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STANDARDIZED FLEXIBLE END PLATE CONNECTIONS FOR STEEL BEAMS

by

GARY J. KRYWIAK



A THESIS

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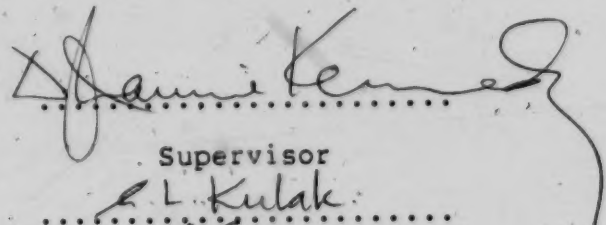
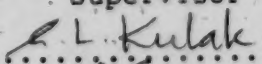
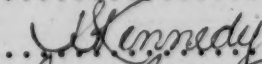
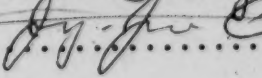
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Abstract

The requirements for the analysis and design of flexible end plate connections include the determination of primary shear capacity considering the effect of connection deformation, determination of the moment developed during connection rotation, and prediction of the maximum unrestrained rotation of a connection. The experimental research reviewed addresses only the last two of these points. Connection design handbooks used in several different countries generally neglect the effect of secondary forces developed when the connection rotates, and propose that flexible end plates be designed to transfer primary shear only; empirical methods are adopted to ensure that the connection has adequate rotational flexibility.

The proposed analysis and design method employs, with modifications, an existing analytical method for evaluating the moment-rotation behaviour of a flexible end plate connection. An extension of this analytical method is used to determine the magnitude and distribution of secondary forces in all components of the connection, for a given rotation. A series of lower bound resistance equations for predicting the primary shear strength of a flexible end plate connection when considering the effect of the secondary forces developed, are proposed.

The three step connection design approach adopted is consistent with Limit States Design philosophy and is used in the development of an interactive computer design program

for standardized connections. Since the connection design model is completely general, the design program will not restrict users to any particular geometric or material properties, as is currently the case in existing standardized connection design manuals. The program will be of value to both structural engineers and steel fabricators.

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List of Symbols

- a = distance between the centreline of the bolt and the assumed location of the prying force resultant
- A_b = nominal cross sectional area of the bolt
- A_w = effective throat area of weld
- b = width of the supported member flange OR length of bearing for a compression bearing specimen
- B = tension force in bolt
- B_i = tension force in bolt at i th level
- B_u = ultimate bearing capacity of the end plate
- C = compressive force in supported beam web
- d = depth of section
- d_b = nominal bolt diameter
- d_h = bolt hole diameter
- d_t = distance from the top of the end plate to the top surface of the flange of the supported beam
- dy = elemental length of the end plate
- e = distance from the center line of the bolt hole to the nearest edge of the end plate
- $e(x)$ = depth from the top or bottom of the supported member to the fibre strained to ϵ_y

E	= modulus of elasticity of supported member
E_p	= modulus of elasticity of end plate
f	= fillet weld leg size
g	= bolt hole gage
G	= elastic shear modulus
h	= total depth of end plate
H	= the horizontal reaction force acting at the bolts in the plane of the undeformed connection
H_i	= the horizontal reaction force acting at the bolts in the plane of the undeformed connection at the i th bolt level
H_{max}	= the maximum horizontal reaction that can develop at the bolts
i	= bolt level of interest
I_p	= moment of inertia of end plate member
j	= number of pairs of bolts below neutral axis of connection
k_{sp}	= coefficient equal to the ratio of the ultimate shear strength of a bolt through the threads, to the ultimate shear strength of the bolt through the gross shank area
k_v	= coefficient equal to the ratio of the ultimate shear strength of a bolt through the gross shank, to the ultimate tensile strength

- k_1 = coefficient relating the maximum bolt deformation to the bearing stress developed between the bolt and the plate
- k_2 = coefficient relating bolt deformation to the bearing stress developed between the bolt and the plate at a load level of 80 percent of the ultimate shear capacity of the bolt
- K_{A1} = the stiffness of the bolts in tension
- K_{A2} = the stiffness of the bolt zone parallel to the undeformed plate
- K_{R1} = the flexural stiffness of the end plate
- l = distance between the inner edge of the bolt hole and the toe of the fillet weld of the undeformed plate OR span of supported member
- l_b = distance from the neutral axis in the connection to the underside of the bottom flange of the supported member when bottom flange bearing is imminent
- l_{bna} = length of end plate below the neutral axis
- M = maximum moment acting in clear span of end plate
- M_{end} = beam end moment induced at support
- $M(x)$ = bending moment at location 'x'
- M_f = factored bending moment
- $M_i(x)$ = internal moment at location 'x'
- M_p = the plastic moment capacity of a rolled shape

- M_{rc} = flexural resistance of coped section assuming no shear force is present
- M_{up} = ultimate flexural capacity of a rectangular section
- n = number of pairs of bolts
- p = width of an end plate tension specimen OR bolt hole pitch for a full size connection
- P = tensile membrane force acting in end plate
- P_{max} = maximum membrane force that can develop in an end plate
- P_{up} = ultimate tensile capacity of the gross section of the end plate
- P_{yp} = yield tensile capacity of the gross section of the end plate
- Q = prying force resultant
- $Q(x)$ = distributed prying force
- t = flange thickness
- t_p = end plate thickness
- t_s = thickness of supporting element
- T = tension force in the supported member web
- T_f = factored tension force in a bolt
- T_i = the nominal tensile force at the web to plate interface at the level of the i th row of bolts

- T_{max} = the limiting tension force that can develop in the web of the supporting member prior to membrane action in the end plate
- T_u = ultimate tensile strength of the net cross section of the end plate
- V = shear force in end plate corresponding to end plate moment M
- V_c = shear force in web of supported member above the top of the end plate
- V_f = factored shear force in a bolt
- V_{ft} = total factored shear force at connection
- V_i = factored shear resistance of connection component at the level of the i th row of bolts
- V_L = longitudinal shear force
- V_{rb} = factored shear resistance of bolt group
- V_{rc} = shear resistance of the web at the coped section assuming no flexural stresses are present
- V_{rm} = factored shear resistance of the web of the supported beam immediately adjacent to the end plate
- V_{rmax} = the ultimate shear strength of a connection when θ is equal to 0 radians
- V_{rp} = factored shear resistance of end plate
- V_{rw} = factored shear resistance of a fillet weld
- $V_{r\theta}$ = the ultimate shear strength of a connection at rotation θ

- V_T = transverse shear force
- V_u = ultimate shear strength of a bolt through the threads
- V_U = ultimate shear strength of a fillet weld loaded by longitudinal forces only
- w = web thickness OR uniformly distributed load
- x = distance from the support to a particular location along the beam
- X_u = ultimate strength of weld metal as rated by the Electrode Classification Number
- α = angle of deformation of end plate assuming hinges at bolt line and web rotate same amount
- α_p = angle of deformation of end plate at which H_{max} is first reached
- α_y = angle of rotation of hinge when the maximum moment attainable first occurs
- α_1 = angle of rotation of the hinge nearest the bolt line
- α_2 = angle of rotation of the hinge nearest the web
- α_3 = angle of rotation of the edge portion of the end plate
- β = an interaction factor derived from tests relating shear and tension forces in a bolt
- Δ = transverse connection deformation
- Δ_{A1} = the deformation of spring A1

- Δ_{A2} = the deformation of spring A2
- Δ_{be} = the bearing deformation of a flexible end plate between the bolt holes and the edge of the connection plate
- Δ_{bemax} = the maximum bearing deformation of a flexible end plate between the bolt holes and the edge of the connection plate
- Δ_{bs} = the shear deformation of a bolt at a load level less than the ultimate shear capacity of the bolt
- Δ_{bsmax} = maximum shear deformation of a bolt
- Δ_e = maximum elastic end plate deformation
- Δ_{ns} = the net section tensile deformation for a flexible end plate
- Δ_{nsmax} = the maximum tensile deformation that can occur at the net section
- Δ_p = maximum tensile deformation of end plate occurring when H_{max} is first reached
- Δ_y = the maximum end plate deformation occurring when the yield moment is attained at the hinges
- ϵ_y = the axial yield strain of the material
- $\epsilon_{max}(x)$ = the maximum flexural fibre strain at a cross-section at location 'x'
- γ = shear strain at shear stress τ
- γ_y = shear strain at shear stress τ_y , ie. yield level shear strain
- π = a constant, 3.14

- ϕ = resistance factor, taken as 0.67 for bolts and for welds, and 0.90 for plate
- $\Phi(x)$ = the curvature at location 'x' along the beam span
- σ = maximum normal stress that can simultaneously be present with shear stress τ
- σ_{bs} = average bearing stress between bolt and plate at a load level less than the ultimate shear capacity of the bolt
- σ_{bsmax} = maximum average bearing stress between bolt and plate
- σ_c = compressive stress at interface of end plate and supported member web
- σ_u = ultimate tensile strength of material
- σ_{ub} = ultimate tensile strength of the bolt
- σ_{up} = ultimate tensile strength of end plate
- σ_{uw} = ultimate tensile strength of supported member web
- σ_y = yield tensile strength of material
- σ_{yp} = yield tensile strength of end plate
- σ_{yw} = yield tensile strength of supported member web
- τ = nominal shear stress
- τ_y = nominal shear stress necessary to cause yielding
- θ = rotation of connection

θ_b = end rotation of the supported member when bottom flange bearing is imminent

θ_{end} = end rotation of the supported member

θ_{max} = maximum attainable rotation for a connection prior to failure

1. Introduction

1.1 General

One of the most important steps in the design, fabrication, and erection of structural steel building frames is the design of the connections. Because there is usually a relatively large number of connections required in common steel frames, the economics of a project can be greatly affected by the time spent in selecting and manufacturing the connections.

To minimize the time required to design and detail connections, some steel fabricators have developed so-called 'shop standards', which are simply handbooks containing a summary of preferred, pre-designed connections suitable for use in standard type steel frames. When developing shop standards, fabricators include connection details most convenient to them, with preferred geometric and material properties. The details are generally simple, uniform, and easy to manufacture. Because they are specified repeatedly by detailers, the standard connection types are often stockpiled, thus streamlining the fabrication process of standard steel frames.

Unfortunately, good sets of shop standards may not be available to all steel fabricators because of the cost to develop them. In some countries, including for example Australia, South Africa, and the United Kingdom, agencies working on behalf of the steel producers, designers,

fabricators, and erectors, have published standardized connection design handbooks that are available industry-wide.

By assisting fabricators in the rapid design of standard connections, these publications encourage industry-wide standardization by tabulating only a limited number of connection details. Opponents to this approach consider that the variation in fabrication equipment, and the individual preferences on such matters as geometric details, make such a high degree of standardization undesirable and not feasible. Ideally, some other means of assisting fabricators in standardized connection design is required.

A prerequisite to the development of standardized or pre-designed connections, is a rational analysis and design procedure to ensure the strength and serviceability of the system. Although most flexible shear type connections common to standard steel frames are designed using simple models, it is recognized that the behaviour of such connections is usually complex. The models have evolved from long standing accepted practice, engineering judgement, and research. Although performance determines whether designs are acceptable or not, connections in the field are rarely loaded above service levels, and the response at overload is generally not well known.

In keeping with the Limit States Design philosophy now in widespread use in Canada, connection resistance should be

determined on such principles to provide adequate but consistent margins of safety. The work of Butler et al. (1972), and Dawe and Kulak (1974), revealed that simple analysis methods, like those based only on elastic response of the connection, can result in connections possessing a considerable range of safety. For these reasons it is desirable to review connection design models and to propose new analysis techniques where necessary.

1.2 Scope and Objectives

The overall objective of this research is to develop a method for the standardization of steel connection design in Canada. To achieve this primary objective, only one type of connection, the flexible end plate, has been investigated. This general method can then be used as a basis for the development of standardized details for other connections.

The work is presented in two parts. The majority of this investigation is found in PART A, where a method is presented for the analysis and design of flexible end plate connections. The method is consistent with the Limit States Design approach. The development of the model is based solely on existing research.

PART B of the research applies the analysis and design model developed in PART A in the development of a method of standardizing connection design. The method does not restrict users to a limited number of connection parameters, as is the case in industry-wide standards currently

available, but allows the individual to specify the parameters as he sees fit.

PART A: PROPOSED ANALYSIS AND DESIGN METHOD FOR THE FLEXIBLE
END PLATE SHEAR CONNECTION .

2. Requirements of Analysis and Design Model

2.1 Background

A flexible end plate connection consists of a rectangular plate symmetrically fastened to the supported member web with fillet welds on both sides of the web, as shown in Figure 2.1. The rectangular plate is fabricated with holes in it so that the field connection can be made by bolting the connection to the supporting member.

Flexible end plate connections are used in steel frame construction to support transversely loaded beams or girders intended to act as simply supported members. Thus, to provide satisfactory performance, the connection must transfer the end shear force from the supported member to the supporting member, allow at most only a negligible end moment to develop in the supported member, and allow virtually unrestrained rotation to occur.

The end rotation of the supported member is accommodated mostly by the out-of-plane deformation of the end plate. Depending upon the relative stiffnesses of the supported member, the connection plate, and the supporting element, some deformation will also occur in the supported and supporting elements. The out-of-plane action causes compatibility or secondary forces to develop in the

connection. These secondary forces induce a moment in the supported beam and in the supporting member. In addition, the secondary forces may adversely affect the shear transfer from the beam to the support.

It is usually assumed that simple shear type connections allow unlimited free end rotation of the supported member to occur. Unfortunately, this is not always the case. Flexible end plate connections have a limiting rotation above which considerable restraint takes place to end rotation of the supported member. This restraint occurs when the bottom flange of the beam comes into contact with the support, due to beam end rotation, resulting in much changed and undesirable behaviour of the connection.

In recognition of this type of connection behaviour, an analytical method intended for use in designing flexible end plate connections should

1. allow for the determination of the shear capacity of the connection in the presence of the secondary forces,
2. allow for the evaluation of the moment-rotation relationship of a flexible end plate connection, and
3. allow for the estimation of the maximum unrestrained end rotation of a member supported by a flexible end plate connection.

2.2 Experimental Studies of Flexible End Plate Connections

Flexible end plate connections have been used for about 20 years. Research on the behaviour of these connections has been done by Kennedy (1966), Kennedy (1969), Sommer (1969), Mansell and Pham (1981), Hafez (1982), and Kennedy and Hafez (1984). (The work done by Mansell and Pham (1981) was not obtainable at the time this report was prepared.) In the literature reviewed, the main objective appears to have been the evaluation of the moment-rotation characteristics of the connections; it is generally assumed that connections designed to transfer shear only will have adequate strength and that this shear strength is unaffected by the moments developed in the connection.

Kennedy (1966) tested a series of identical cantilever beams supported by flexible end plates with varying moment-shear ratios by applying a load at different lever arms. The moment-rotation characteristics for the different moment-shear ratios were essentially the same. Kennedy suggested that the flexibility of the connections was independent of the shear force and, hence, that the shear strength was not significantly affected by the moment. None of the connections, even those with high shear-moment ratios, failed in shear. The results appear to substantiate the common practice of designing 'simple' end connections for shear only, neglecting the end moments developed in them. While this may appear to be justified considering the relatively small moments developed, and the limited rotation

required, the effect of the rotation on the ultimate shear strength was not assessed.

The main objective of Sommer's (1969) test program, an extension of Kennedy's work, was to assess the relative effects that geometric parameters have on the connection behaviour. Sommer determined that, in general, flexible end plate connections become stiffer flexurally as the thickness of plate is increased, as the depth of the connection is increased, or as the bolt hole gage is decreased. It was reported that, for the range of flexible end plate connections tested, the end moment developed ranged from 4 percent to 25 percent of the yield moment capacity of the supported member, with the average being 9 percent. Sommer concluded from his observations that the common practice of designing the connection components, bolts and welds, to transmit primary shear only, is satisfactory. However, Sommer warns of the potential problem of premature bolt fracture in deep connections, due to secondary forces induced in the bolts at the top of such connections when large outward movements occur as a result of supported beam end rotations.

The connections, tested as cantilever connections, failed generally in the connection plate at rotations considerably greater than those encountered for the 'simply supported' ends of beams commonly used in steel construction. None of the 20 tests failed in shear. Coupled with Kennedy's previous tests with high shear-moment ratios,

this would indicate that at practical values of end rotations the degradation of shear capacity due to the rotation may not be large. It is noted that an assessment of this degradation was not made.

Indeed, beams designed to carry large shears will likely have only small end rotations as the spans will not be long. However, in any connection, regardless of the supported member length, the bolts and welds may be selected to carry only the shear that is present. Therefore, under the factored loads, these connection components could reach their capacities prematurely, because no consideration was made of the effects of end rotation on those capacities. Moreover, as shown in Table 2.1, in Sommer's tests the average shear on the bolts at failure had a maximum value of 36 percent of the nominal shear capacity through the threads and the average shear on the welds did not exceed 26 percent of the ultimate capacity. Even though the failures, which occurred most often in the connection plate, happened at connection rotations considerably greater than those commonly encountered in steel frames, this condition indicates that some reduction of the ability of these connections to carry shear must occur between no rotation and ultimate rotation levels, as postulated in Figure 2.2.

Hafez (1982) (see also Kennedy and Hafez (1984)) developed an analytical method for predicting the moment-rotation characteristics of flexible end plate connections up to and including the rotation at which the

bottom flange bears against the support. The predicted relationship accorded well with test results.

To provide a complete picture of the behaviour of flexible end plates, it was therefore considered necessary to review, and possibly modify, the method of Hafez, and of prime importance to assess quantitatively the effect of the rotation on the shear strength of the connection.

2.3 Connection Handbook Analysis and Design Procedures

Even though the Australian Institute of Steel Construction (AustISC, 1978), the American Institute of Steel Construction (AISC, 1980), the Canadian Institute of Steel Construction (CISC, 1980), the British Constructional Steelwork Association (BCSA, 1982), and the South African Institute of Steel Construction (SAISC, 1982) present unique resistance equations for the design of flexible end plate connections, their methods are, for the most part, similar.

The CISC and the AISC publications are not truly standardized connection design handbooks. They are included, however, because they represent the major industry-wide documents of this type available in North America. The connection design information provided in these handbooks is provided (among other reasons) to assist individual fabricators in developing their own shop standards.

In the design for strength, all of these agencies assume that the connection is subject only to a vertical shear force, uniformly distributed throughout the

connection, determined assuming the supported member to have ideal hinged supports. The limiting strength of a flexible end plate connection is then based on the component having the minimum resistance: that of the bolt group, the end plate, the fillet weld, or the supported beam web. The effect of coping of the supported beam should also be investigated.

The shear strength of the bolt group is limited to the smallest of the following three values: the shear strength of the bolts, the bearing strength of the end plate, and the bearing strength of the support. Normally, threads are assumed to exist in the shear plane when evaluating the bolt shear strength for flexible end plate connections.

The AustISC recommends that for long connections, those with more than six rows of bolts, a reduction factor be applied to the calculation of the bolt group shear strength. This reduction factor is intended to account for uneven loading of the bolts, which is common to long connections of tension members. This consideration does not seem appropriate for shear type connections where the shear transfer to the bolts is likely more uniform. None of the other publications recommend that a reduction factor be applied to bolt shear strength for long connections.

The AustISC recommends a lower allowable bearing strength for the end plate than for the supporting element. This is to account for the horizontal shear forces induced in the bolts due to the horizontal connection movement that occurs during beam end rotation, and for the relatively

small loaded edge distances in the end plates as compared to the supporting material. Since the deformations in long connections, due to beam end rotation, can be relatively large, the secondary forces developed also can be relatively large. The reduced shear strength assigned to bolts in long connections, rationally, should be assigned to account for the secondary forces and not to account for uneven vertical shear transfer. None of the manuals directly considers the effect of the development of tension and horizontal shear forces in the bolts.

In the four remaining handbooks, the bearing capacity of the support and of the connection end plate is evaluated using only one bearing capacity expression, neglecting the effects of horizontal connection movement. The SAISC manual states, however, that if bolt hole deformations due to high bearing stresses are undesirable, then the allowable bearing stress should be reduced by 25 percent in both the support and the plate.

In the AustISC handbook, the shear capacity of the end plate is based on its gross section while the BCSA manual uses the net section through the bolt line. The AISC, CISC and SAISC manuals do not cover this point.

The longitudinal shear strength of the fillet welds at the supported beam to end plate junction as used in the AustISC, AISC, and BCSA Handbooks is based on effective weld lengths equal to the specified length minus two times the fillet leg size, to account for the reduced weld profile at

the start and stop positions. The SAISC approach appears identical to that used by the BCSA. The CISC handbook simply uses the specified weld length. None of the five handbooks considers the presence of transverse shear components, induced in the fillet welds due to end rotation of the supported beam, when evaluating the strength.

In the AustISC manual, the allowable average shear stress in the web of beams with top flange copes is limited to 89 percent of that allowed for uncoped members and is limited to 81 percent when both the top and bottom flanges are coped. The AustISC provides guidelines for evaluating the reduced shear capacity of webs with certain size copes only.

The approach for evaluating coped member web shear strengths in both the BCSA Handbook and the SAISC Handbook is similar to the Australian approach, but it is not explicitly covered in the Canadian or American handbooks. All of the handbooks deal with the shear strength of uncoped webs in the same way, and assume the shear strength of the web is available only over the length of the connection.

Of all of the five handbooks, only the AustISC provides recommendations regarding the design eccentricity that should be assumed for the flexible end plate when designing the supporting member. In recognition of the partial fixing moment that develops at the connection, the AustISC suggests the design eccentricity be equal to 100 mm greater than the geometric eccentricity. This requirement is suggested for use only when the support is a column, and no

recommendations are given for the case where the supporting member is a beam or girder.

To ensure that the connection has adequate flexibility and to preclude the bottom flange of the supported member from colliding with the support, the AustISC has applied the recommendations of Kennedy (1969). Certain geometric constraints must be met. With a maximum expected supported beam end rotation of 0.030 radians, based on the assumption that the maximum bending moment is the yield moment, and with the further assumptions that placement of the end plate on the web of the supported member is typical and that all beam end rotation occurs about the bottom fibre of the end plate, then free rotation occurs if the connection plate length does not exceed 33 times its thickness. This geometric constraint does not, of course, apply if the bottom flange is coped.

The BCSA handbook follows the AustISC procedure for attaining adequate rotation. The other handbooks have no specific requirements.

To ensure flexibility of the connection, the end plate must pull away from the supporting member at the top. The AustISC handbook states that the ratio of the bolt gage to plate thickness must be between 11 and 14 to ensure this. By providing sufficiently flexible end plates, the secondary forces developed at the connection, and therefore the moments developed at the connection, are limited. The other handbooks give limits on the end plate thickness of 6 mm and

10 mm and on the gage of 89 mm and 150 mm.

Deeper connections tend to induce greater secondary forces (Sommer 1969). Only the SAISC specifically recommends that flexible end plate connections can be used on members as deep as 2000 mm.

Of the five connection handbooks, only the CISC and the AISC publications provide guidelines for the design of flexible end plate connections for slip-resistant conditions. The flexible nature of this connection, as observed in tests (Sommer 1969, and Hafez 1982), is not likely compatible with slip-resistant bolting procedures because of the deformations that can occur in the connection. Furthermore, it is possible that the connection will slip at load levels less than those assumed by the designer, because the designer, when sizing the connection, is not likely to consider the presence of the secondary shear and tension acting in the bolts above the neutral axis of the connection.

In general, the handbooks discussed here evaluate the strength of the flexible end plate as if the connection behaved as a perfect hinge. Also, the analysis and design approach adopted by all of the handbooks seems to ignore the fact that significant inelastic response occurs in the flexible end plate connection even at service level loads, as observed by both Sommer (1969) and Hafez (1982). Even though the CISC Handbook evaluates the resistance of the connection components using a Limit States Design standard

(CAN3-S16.1-M78, CSA 1978), it does not deal with the analysis of the design forces in a compatible way. Some empirical rules are given in the handbooks to limit the secondary forces that develop in these connections. However, no general design approach to account for the secondary forces is given. To step beyond the bounds of empiricism and provide greater consistency, an analytical approach is needed that takes into account the effect of these secondary forces that develop due to the deformation of the connection.

2.4 Proposed Approach

The primary shear force for a flexible connection can be directly determined on the basis that the shear distribution in the supported member is not significantly affected by the relatively small end moments that exist. In the extreme case of a beam supporting a uniformly distributed load, as shown in Figure 2.3, even when the end moments are unequal, and one end moment is taken as 25 per cent of the maximum span moment and the other is taken as zero, the end shear is changed by only 6 percent. Because the magnitude and distribution of secondary forces that develop in flexible end plate connections depend on the connection geometry and can affect the primary shear strength of the connection, the analysis and design procedure will generally need to be iterative.

The suggested procedure is:

1. select an initial end plate configuration, using the simple vertical shear design method proposed in the steel industry handbooks, neglecting secondary effects,
2. evaluate the moment developed at the rotation associated with the factored load on the supported member,
3. check to ensure that uninterrupted connection rotation is possible at this load,
4. determine the primary shear capacity considering the secondary effects, and
5. iterate as necessary, as depicted in the flow chart of Figure 2.4.

Table 2.1 Summary of Average Stresses and Forces in Sommer
(1969) Connection Tests at Failure

Test No.	Ult. Load (kips)	Plt. Size Th. x Dep. (inches)	Plt. Yld. Strength (ksi)	No. of Bolts	Fillet Size (inches)	PLATE Avg. Str. Yld. Str.	BOLTS ¹ Avg. Sh. Sh. Cap.	WELD ² Avg. Str. Ult. Str.	Location Of Rupt.
5	26.9	1/4 x 15	49.3	10	1/4	0.073	0.120	0.110	HAZ Plt.
6	28.5	1/4 x 9	49.3	6	5/16	0.130	0.220	0.155	HAZ Plt.
7	33.3	1/4 x 12	49.3	8	5/16	0.110	0.190	0.136	HAZ Plt.
8	29.8	1/4 x 15	49.3	10	5/16	0.081	0.137	0.097	HAZ Plt.
9	29.9	1/4 x 18	49.3	12	5/16	0.067	0.114	0.081	HAZ Plt.
10	27.0	3/8 x 8	57.2	6	1/4	0.070	0.206	0.184	HAZ Plt.
11	33.0	3/8 x 12	57.2	8	1/4	0.064	0.189	0.168	HAZ Plt.
12	21.5	3/8 x 15	57.2	10	1/4	0.033	0.099	0.088	HAZ Plt.
13	47.0	3/8 x 9	57.2	6	5/16	0.120	0.359	0.256	HAZ Plt.
14	57.3	3/8 x 12	57.2	8	5/16	0.110	0.328	0.234	HAZ Plt.
15	49.1	3/8 x 15	57.2	10	5/16	0.076	0.225	0.160	Bolt
16	54.7	3/8 x 18	57.2	12	5/16	0.071	0.209	0.149	Bolt
17	22.5	1/4 x 12	49.3	8	5/16	0.076	0.129	0.092	HAZ Plt. ⁴
18	36.7	1/4 x 15	49.3	10	5/16	0.100	0.168	0.120	HAZ Plt.
19	51.7	1/2 x 12	52.6	8	5/16	0.082	0.296	0.210	HAZ Plt.
20	47.0	1/2 x 15	52.6	10	5/16	0.060	0.216	0.157	HAZ Plt.
25	17.0	1/4 x 12	49.3	8	1/4	0.057	0.098	0.089	HAZ Plt.
26	13.8	1/4 x 9	49.3	6	1/4	0.062	0.106	0.075	HAZ Plt.
27	7.2	1/4 x 9	49.3	6	3/16	0.033	0.055	0.065	HAZ Plt.
28	8.3	1/4 x 6	49.3	4	3/16	0.056	0.095	0.113	HAZ Plt.

1 A325 3/4 inch Diameter Bolts, ULTIMATE STRENGTH = $0.60 \times 0.70 \times \sigma_{ub} \times A_b$

2 E7018 Electrodes,

ULTIMATE STRENGTH = $0.66 \times X_u \times f \times h \times 2\sqrt{2}$

3 HAZ Plt. : Heat Affected Zone of End Plate.

4 BL Plt. : Bolt Line of End Plate.

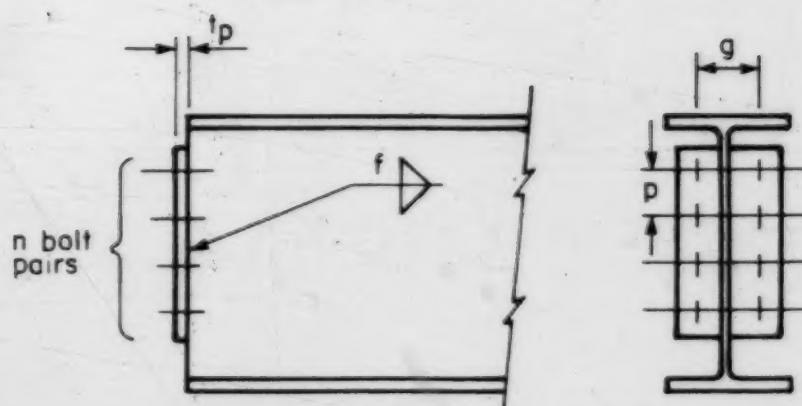


Figure 2.1 Flexible End Plate Connection Detail

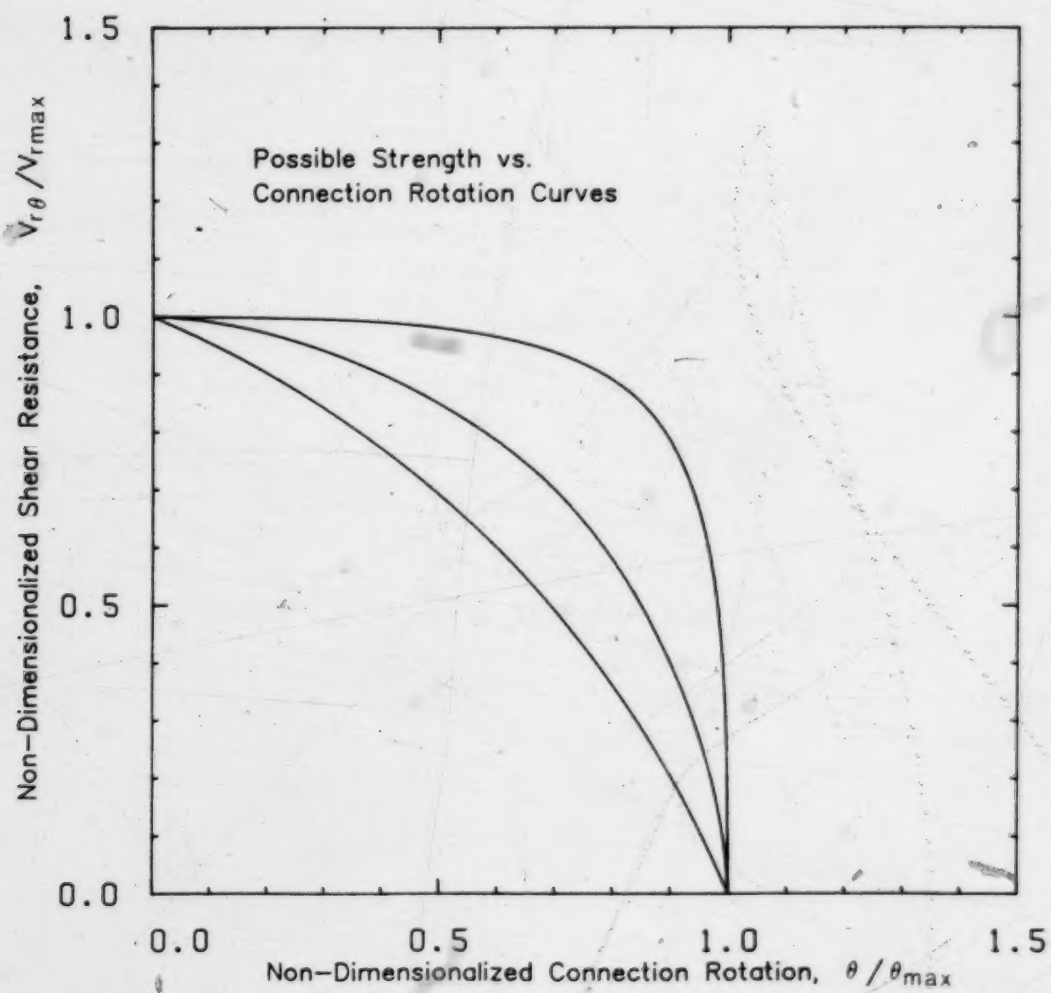


Figure 2.2 Possible Relationship of Flexible End Plate Shear Resistance to the Connection Rotation

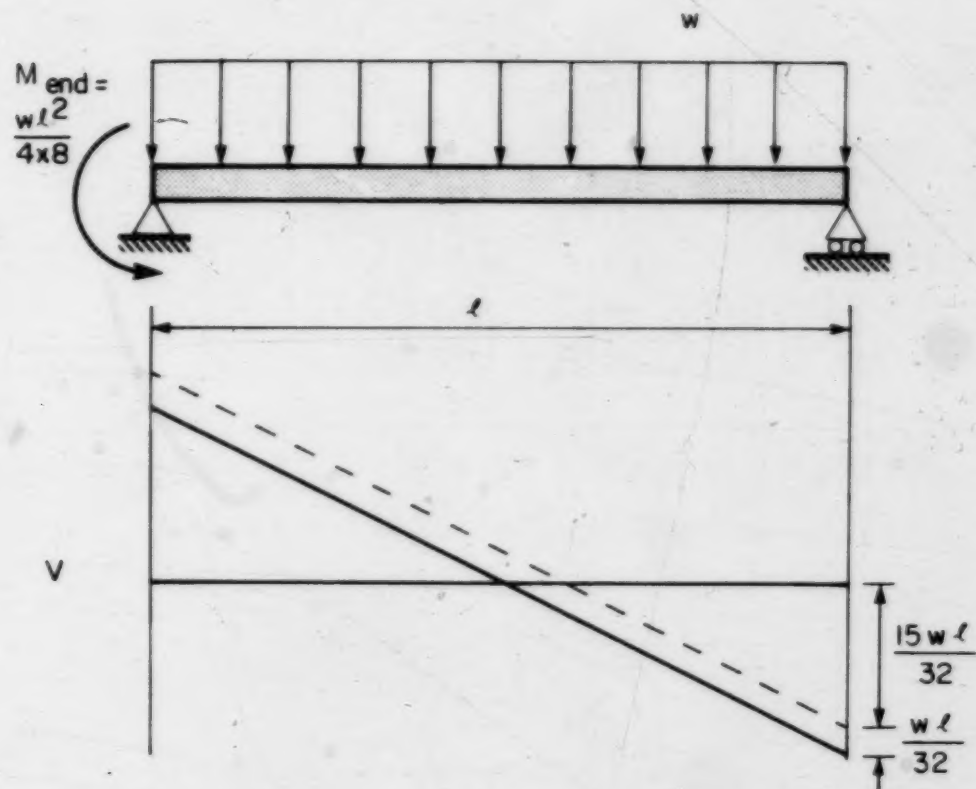


Figure 2.3 The Effect of Relatively Small Support Moments on the Shear Distribution in Beams

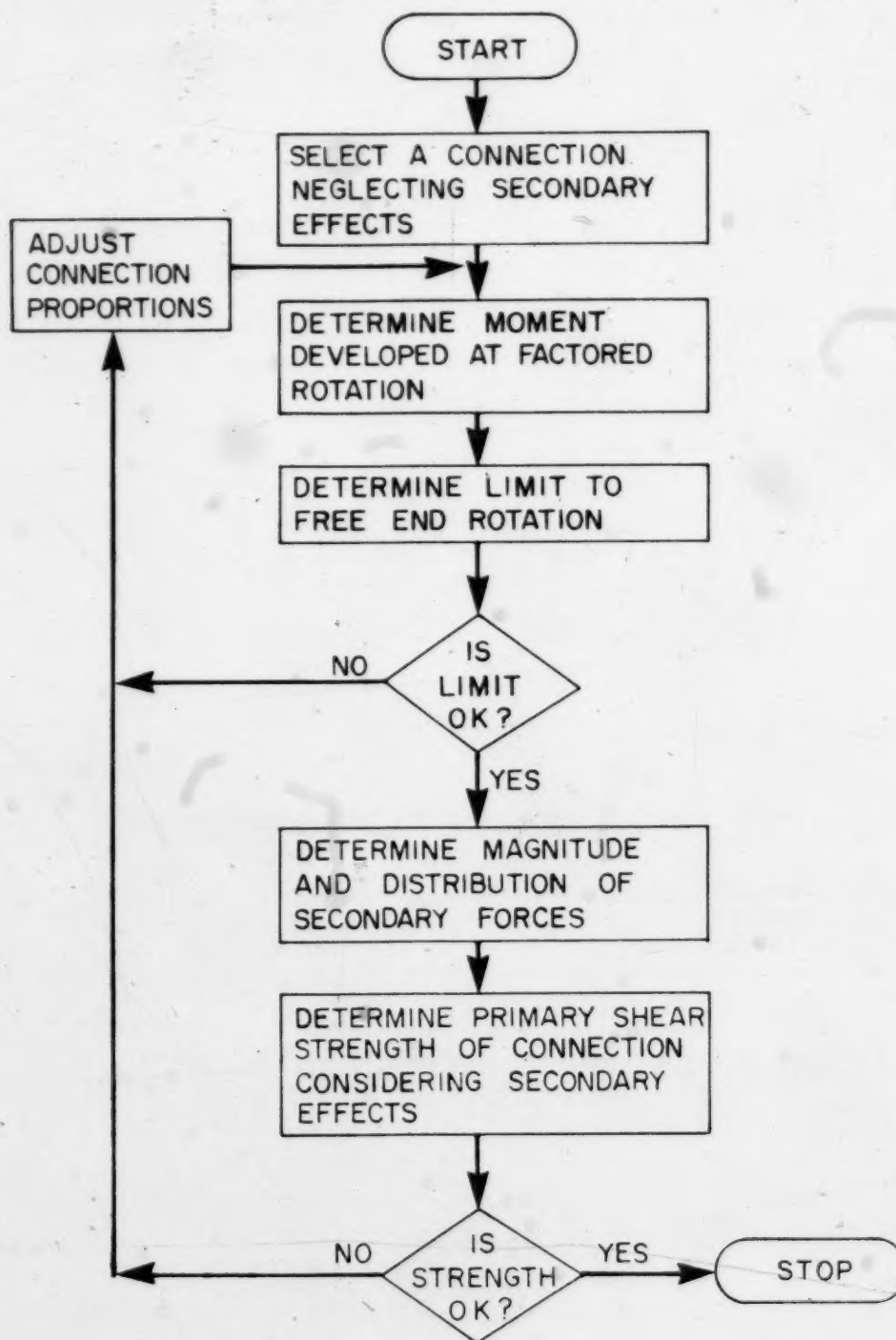


Figure 2.4 Design Process for a Flexible End Plate
Connection

3. Evaluation of the Moment-Rotation Relationship of a Flexible End Plate Connection

3.1 Introduction

Sommer (1969) and Hafez (1982) investigated experimentally the moment-rotation characteristics of flexible end plate connections. Sommer evaluated various connection geometries and developed an empirical standardized equation from which the moment-rotation relationship could be predicted.

Hafez developed an analytical method to predict the moment-rotation relationship that agreed well with experimental results. This method is extended here by removing some of the simplifications introduced by Hafez.

3.2 Background and Description of Analytical Method

Hafez observed, as had others, that the deformation of the connection varied linearly over its depth and that the location of the neutral axis moved downward as the connection rotation increased, indicating that the stiffness in the compression region differs from that in the tension region. Hafez next developed analytical load-deformation relationships for end plate details, loaded either in tension or compression, that correspond closely with test results.

To calculate the moment developed for a given rotation, a position of the neutral axis is first assumed and the

portion of the connection on either side of the axis is divided into a number of elements. For small rotations, the neutral axis is at about mid-height and moves downward with increasing rotation. With the deformations of each element now known, the forces developed in them based on the load-deformation relationships established previously are determined, as shown in Figure 3.1. These elemental forces are summed, and the position of the neutral axis is modified until the sum is zero. The sum of the moments of the elemental forces for this position gives the moment in the connection for the assumed rotation.

3.3 Load-Deformation Relationships of Connection

3.3.1 Original Hafez Relationships

3.3.1.1 Tension Region

For the tension region of the end plate, Figure 3.2, Hafez proposed two expressions to describe different portions of the load-deformation response. When the deformation is elastic and small, the relationship between tensile force and plate deformation is given by:

$$[3.1] \quad T = \frac{24 E_p I_p}{l^2} \tan \alpha$$

But $\tan \alpha$ equals Δ/l and for a unit width of plate I_p equals $t_p^3/12$. Therefore:

$$[3.2] \quad T = \frac{2 E_p t_p^3}{l^3} \Delta$$

It is assumed that the moment-curvature relationship for the rectangular section is bilinear and that the maximum moment is reached at the ultimate tensile strength. From the free body diagram of Figure 3.2, the limiting value of the tensile force for this behaviour can be expressed as:

$$[3.3] \quad T_{\max} = \frac{4 M_{up}}{l}$$

where

$$[3.4] \quad M_{up} = \frac{\sigma_{up} t_p^2}{4}$$

Although it is common to base the moment developed at plastic hinges on the yield strength of the material, Hafez (1982) found that using the ultimate tensile strength gave an analysis more closely fitting test results. This was attributed to the occurrence of significant strain-hardening at the hinge locations.

Combining Equations [3.2], [3.3], and [3.4] gives the maximum plate deformation for which Equation [3.2] is valid:

$$[3.5] \quad \Delta_e = \frac{\sigma_{up} l^2}{2 E_p t_p}$$

For the second stage of behaviour, it is assumed, based on test observations, that plastic hinges have developed

adjacent to the toe of the fillet weld and at the inner edge of the bolt holes. Outward movement of the plate may be large enough so that significant load can be carried in tension, that is by membrane action, as well as by flexural action. From the free body diagrams shown in Figure 3.3 the following relationship can be developed:

$$[3.6] \quad T = 2 P \sin \alpha + \frac{4M}{l} \cos^2 \alpha$$

Assuming the flexural-tension interaction expression for rectangular sections (see "Plastic Design in Steel", ASCE (1971)) is taken as:

$$[3.7] \quad \frac{M}{M_{up}} + \frac{P^2}{P_{up}^2} = 1.0$$

and combining Equations [3.6] and [3.7] gives:

$$[3.8] \quad T = 2 P \sin \alpha + 4 \frac{M_{up}}{l} \cos^2 \alpha - \frac{4 M_{up} P^2}{l P_{up}^2} \cos^2 \alpha$$

Setting the partial derivative of T with respect to P equal to zero results in the value of P being given as:

$$[3.9] \quad P = \frac{l P_{up}^2 \sin \alpha}{4 M_{up} \cos^2 \alpha}$$

Substituting Equation [3.9] into [3.8] gives:

$$[3.10] \quad T = \frac{l P_{up}^2}{4 M_{up}} \tan^2 \alpha + \frac{4 M_{up}}{l} \cos^2 \alpha$$

It was observed after testing end plate specimens in tension that extremely large strains appeared in the plate at plastic hinge locations and that strains in between the hinge locations were considerably less. For this reason, Hafez suggested that the maximum moment be based on the ultimate tensile stress, σ_{up} , but that the maximum axial load in the plate be based on the yield strength, σ_{yp} . The final analytical expression presented by Hafez for determining the inelastic tensile load-deformation response of a flexible end plate is:

$$[3.11] \quad T = \frac{l P_{yp}^2}{4 M_{up}} \tan^2 \alpha + \frac{4 M_{up}}{l} \cos^2 \alpha$$

or, in terms of Δ :

$$[3.12] \quad T = \frac{P_{yp}^2}{4 M_{up} l} \Delta^2 + \frac{4 M_{up} l}{(l^2 + \Delta^2)}$$

The first part of this expression is the contribution due to the membrane action and the second part is the contribution due to flexural action. When the deflection is zero there is no membrane contribution. Equation [3.12] implies that there is a maximum flexural contribution when the deflection is zero, which is clearly not the case. This anomaly arises from the assumption of initial full plasticity. Equation [3.12] is to be used only at deformations greater than that given by Equation [3.5]. As

shown by Hafez, the two behavioural modes are in reasonable agreement with the test behaviour provided that initially the flexural behaviour is sufficiently stiff so that the projected elastic line intersects the curve described by the second mode.

3.3.1.2 Compression Region

In the compression region, deformations due to beam end rotations occur in the web of the supported beam (this condition is most severe when the support element is relatively stiff), as shown in Figure 3.4.

Hafez conducted compression load-deformation tests on end plate specimens of varying lengths, with the web plate made wider than the end plate. These tests were conducted in order to determine the influence of the supported member below the connection plate on the strength and deformation characteristics of the compressed region. Empirical elastic and inelastic relationships were developed relating the deformation to the average stress in the supported beam web at 25 mm from the connection plate surface. These relationships consider a 45 degree spreading of the load from the ends of the end plate, as shown in Figure 3.5.

For the material tested, Hafez's proposals in SI units are:

$$[3.13] \quad \sigma_c = .543 \Delta$$

$$\text{when} \quad 0 < \sigma_c < 166 \text{ MPa}$$

and

$$[3.14] \quad \sigma_c = -146 + 12.3\Delta - 330\Delta^{1/2} + 727\Delta^{1/3}$$

when $166 \text{ MPa} < \sigma_c$

3.3.2 Modifications to the Hafez Relationships

3.3.2.1 Tension Region

A comparison of test results with the analytical tensile load-deformation relationships given by Equations [3.1] and [3.11] were in good agreement except for angles of deformation, α , less than about 0.1 radians and greater than about 0.3 radians. For angles less than 0.1 radians, the discrepancy was attributed to the assumption that the moment-curvature behaviour was elasto-plastic up to M_{up} . For angles greater than 0.3 radians, Kennedy and Hafez (1984) showed that the discrepancy can be attributed to the assumption that the gross section of the end plate was available to resist membrane forces. Kennedy and Hafez instead proposed that the net section strength limits the magnitude of these forces. The effect of these discrepancies on the development of moment-rotation relationships is not significant, as will be shown in Section 3.4.

Kennedy and Hafez (1984) rationalized that the maximum developable membrane force in the end plate is not limited by the tensile yield strength of the gross section but by that of the net section. They modified Equation [3.11] to

give:

$$[3.15] \quad T = \frac{P_{yp}^2 I}{4 M_{up}} \left(\frac{P - d_h}{P} \right)^2 \tan^2 \alpha + \frac{4 M_{up}}{I} \cos^2 \alpha$$

This expression gave better correlation with test results at angles greater than 0.3 radians than Equation [3.11]. However, modifying the original equation in this way implies that the correct flexural-tension interaction expression is:

$$[3.16] \quad \frac{M}{M_{up}} + \frac{P^2}{P_{yp}^2} \left(\frac{P - d_h}{P} \right)^2 = 1.0$$

This implies that the plastic hinges form at the net section of the end plate for the axial response and at the gross section for the flexural response. This, however, realistically cannot be the case, and in fact the hinge lines were observed to always form through the gross section (Hafez 1982). Equation [3.7] with $P_{up} = P_{yp}$ as originally used by Hafez (1982) is considered correct, provided that a limit on the membrane force is imposed.

Although no fractures occurred in the area of the bolts in the Hafez tests, the membrane force in the end plate is limited by either the net section tensile capacity, the bolt shear strength, or the plate bearing capacity, as shown in Figure 3.6. The limiting connection deformation, however, may not be associated with the bolt zone and Hafez (1982) observed fractures in 28 T-section specimens at the fillet

weld location in the plate.

From the free body diagram of Figure 3.6, the force P that can develop in the end plate is:

$$[3.17] \quad P = \frac{2M}{l} \sin \alpha + \frac{H}{\cos \alpha}$$

Combining this with the flexural-membrane interaction expression (Equation [3.7] with $P_{up} = P_{yp}$) gives, in quadratic form, the maximum value of P as:

$$[3.18] \quad P_{max} = \frac{2 M_{up}}{l} \left(1 - \frac{P_{max}^2}{P_{yp}^2} \right) \sin \alpha + \frac{H_{max}}{\cos \alpha}$$

where the force H_{max} is the smallest of the following values: the bolt shear capacity, the net section tensile capacity, and the end bearing capacity. The equations to predict these limiting loads as given in CSA Standard CAN3-S16.1-M78 (CSA 1978), without resistance factors, are respectively:

$$[3.19] \quad V_u = 0.60 \times 0.70 A_b \sigma_{ub}$$

$$[3.20] \quad T_u = (p - d_h) t_p \sigma_{up}$$

$$[3.21] \quad B_u = e t_p \sigma_{up} \leq 3 d_b t_p \sigma_{up}$$

In Equation [3.19] the factor 0.70 accounts for the case when the shear plane passes through the threads, as is likely to be the case for these connections. Because of the

small end distances used in end plate connections the upper limit on bearing strength in Equation [3.21] is unlikely to govern.

Modifications of Equations [3.19] and [3.20] could be considered as they may over-estimate the maximum force developable. The shear resistance of the bolts to the force H , as given by Equation [3.19], is reduced because of the tensile force developed in the bolts (Chesson et al. 1965) and because of the primary shear acting on the end connection. The tensile resistance as given by Equation [3.20] does not take into account any reduction due to the primary shear on the connection.

Figures 3.7 and 3.8 show for two typical end plate connection details the membrane load-deformation relationships based on the equation used by Hafez (1982), on the equation used by Kennedy and Hafez (1984), and on equations proposed herein. The connection details considered have the following material and geometric properties:

$$\begin{aligned} \sigma_{yp} &= 300 \text{ MPa} \\ \sigma_{up} &= 450 \text{ MPa} \\ \sigma_{ub} &= 825 \text{ MPa} \\ w &= 10 \text{ mm} \\ p &= 75 \text{ mm} \\ e &= 30 \text{ mm} \\ f &= 6 \text{ mm} \\ d_b &= 20 \text{ mm} \\ t_p &= 6 \text{ mm or } 12 \text{ mm} \\ g &= 100 \text{ mm or } 150 \text{ mm}, \end{aligned}$$

/ From these two figures, it is seen that both the Hafez and the Kennedy and Hafez equations give greater values than Equation [3.18] for relatively large deformations when H_{max}

is limited by the bearing capacity of the plate or by the shearing capacity of the fasteners (a similar condition could likely be developed where the net section capacity of the connection plate limits the value of H_{max}).

For small deformations, Equation [3.18] gives larger values for P because it is assumed that H_{max} acts without any deformation. However, Fisher (1965), Wallaert and Fisher (1965), Kato and Aoki (1970), Bahia and Martin (1980), Owens et al. (1981), and Frank and Yura (1981) all show that the ultimate forces are attained only with substantial deformations.

In Figures 3.7 and 3.8 it is seen that the maximum value of P given by Equation [3.18] is not significantly larger than the value of:

$$[3.22] \quad P_{max} = \frac{H_{max}}{\cos \alpha}$$

Using this value as an approximation for P_{max} and setting this equal to that given by Equation [3.9] but with P_{up} replaced with P_{yp} , the angle of deformation when H_{max} is reached is given by:

$$[3.23] \quad \alpha_p = \tan^{-1} \left(\frac{4 H_{max} M_{up}}{l P_{yp}^2} \right)$$

or, in terms of deformation:

$$[3.24] \quad \Delta_p = \frac{4 H_{max} M_{up}}{P_{yp}^2}$$

These limits are marked on Figures 3.7 and 3.8. Equation [3.11] would be considered valid below this deformation and Equation [3.18] or [3.22] would be valid above this deformation. Combining Equations [3.22], [3.6], and [3.7], with P_{up} replaced with P_{yp} , gives:

$$[3.25] \quad T = 2 H_{max} \tan \alpha + \frac{4 M_{up}}{l} \cos^2 \alpha - \frac{4 M_{up} H_{max}^2}{l P_{yp}^2}$$

or, in terms of deformation:

$$[3.26] \quad T = \frac{2 H_{max}}{l} \Delta + \frac{4 M_{up} l}{(l^2 + \Delta^2)} - \frac{4 M_{up} H_{max}^2}{l P_{yp}^2}$$

Figures 3.9 to 3.13 compare three analytical methods with the T-section test results of Hafez. For angles of deformation greater than about 0.2 to 0.3 radians, the Hafez relationship (Equation [3.11]) overestimates the tensile force developed. No limit other than the yielding of the gross section was placed on the maximum membrane force developable. Equation [3.15] proposed by Kennedy and Hafez, which assumes membrane forces are limited by the net section strength, and which assumes that the plastic hinge lines form through the hole when considering the axial forces, tends to underestimate the results. By using Hafez's Equation [3.11] when H is less than H_{max} and Equation [3.25] when H equals H_{max} , a better prediction of the load-deformation response is obtained. Although in the

Hafez T-section tests none of the specimens failed by shear of the bolts, fracture of the net section, or tear out, it is probable that the limiting horizontal force, H_{max} , was near to being reached. This conclusion is corroborated by the significant deformations that were observed for the governing mode (Hafez 1982).

3.3.2.2 Compression Region

The empirical equations developed by Hafez (Equations [3.13] and [3.14]) for the load-deformation of the compression region are strictly valid only for steels having the same stress-strain relationships as those used by Hafez. Owens et al. (1981), investigating the bearing characteristics of bolted plates of two different ultimate strengths, found load-deformation responses similar to that of Hafez, and further proposed that the bearing stresses developed for a given deformation were linearly related to the ultimate strength of the plate.

Because of the very close resemblance between the end plate compression specimen tests conducted by Hafez and the bolt bearing tests conducted by Owens et al., as shown in Figure 3.14, it is proposed that Equations [3.13] and [3.14] be modified for general use by multiplying them by the ratio of the ultimate strength of the plate being analyzed to that tested by Hafez (442 MPa or 64.1 ksi). (However, for the initial linear response, the relationship is probably related to elastic properties.) Equations [3.13] and [3.14] therefore become:

$$[3.27] \quad \sigma_c = 543 \left(\frac{\sigma_{uw}}{442} \right) \Delta$$

when $0 < \sigma_c < 166 \left(\frac{\sigma_{uw}}{442} \right) \text{ MPa}$

and

$$[3.28] \quad \sigma_c = \left(\frac{\sigma_{uw}}{442} \right) (-146 + 12.3\Delta - 330\Delta^{1/2} + 727\Delta^{1/3})$$

when $166 \left(\frac{\sigma_{uw}}{442} \right) \text{ MPa} < \sigma_c$

3.4 Comparison of Moment-Rotation Analysis With Test Results

In Figures 3.15 through 3.22, the analytical relationships developed by Hafez, Kennedy and Hafez, and as proposed herein are compared with the Hafez (1982) test results, while in Figures 3.23 to 3.42 they are compared to the Sommer (1969) results. Each curve uses a different combination of the load-deformation relationships. The analytical curves shown as the Proposed Analysis were developed using Equations [3.2], [3.12], and [3.26] in the tension zone, and Equations [3.27] and [3.28] in the compression zone. The Kennedy-Hafez Analysis curves were developed using Equations [3.2] and [3.15] in the tension zone, and Equations [3.13] and [3.14] in the compression zone. The Hafez Analysis curves were developed using Equations [3.2] and [3.12] in the tension zone of the connection, and using Equations [3.13] and [3.14] for compression zone analysis. The analytical curves are

terminated where they predict bearing of the bottom flange of the beam against the supporting member to occur. This point is indicated by a short transverse line at the end of the analytical curves, and this phenomenon will be discussed in detail in Chapter 4. In developing the analytical curves, the specimens were divided into 100 equal elements over the depth of the connection. The computer program used to generate the curves is given in Appendix A.

All three of the analyses predict the slope of the curves reasonably well. Table 3.1 lists test-to-predicted values of the moment and of the corresponding rotation developed in the connections at the instant that bearing of the supported member on the support occurs. Using the Kennedy-Hafez, analysis the mean value of the test-to-predicted ratio for the moment is 1.18, with a coefficient of variation of 0.11. For the proposed method, the mean value for the ratio is 1.06, with a coefficient of variation of 0.10. The better prediction by the proposed method is attributed to the fact that it takes into account more rationally the limit on the membrane force developable in the tension zone of the connection and because it considers more rationally the compressive forces developable at the bottom of the connection. The comparison of rotations is discussed Chapter 4.

At relatively small rotations, less than about 0.01 radians, the analyses overestimate the moment. As reported by Hafez (1982), this is due to the fact that the tension

behaviour has been modelled as elasto-plastic without allowing for gradual yielding.

No deformations of the web of the beam, the fillet welds, or the bolts were considered in the analysis. In the T-section tests, these deformations were relatively small. Butler and Kulak (1971) found for transversely loaded 6.4 mm fillet welds that the ultimate deformation was between 0.5 and 1.0 mm, which is relatively small as compared to total deformations of the entire end plate assembly observed to be up to 30 mm (Hafez 1982). Rumpf and Fisher (1963) and Sterling et al. (1965) found that elongations of bolts loaded up to 95 percent of the tensile capacity rarely exceeded 1 mm. This substantiates the assumption used, that is, that the deformation takes place chiefly in the end plate itself.

In both the Sommer (1969) and Hafez (1982) tests, the end plate connections were connected to a heavy column flange, as shown in Figure 3.43. Figures 3.44 through 3.52 show a variety of other details. Where the supporting member is less rigid than that tested, it is likely that the end moments developed will be less. Coping of the beam web above or below the connection makes the connection behaviour more flexible. Depending on the amount of increased flexibility, this may significantly reduce the moment predicted by any of the analyses.

Although Kennedy (1969) reported that large primary shear forces did not affect the moment-rotation behaviour of

flexible end plate connections, which is substantiated within the limits of the comparisons here with the Sommer (1969) and Hafez (1982) tests, it is possible that very large primary shears might reduce the end moments developed in a manner similar to the condition observed in steel beams carrying large shear forces and bending moments simultaneously (ASCE 1971).

It appears, therefore, that predictions of moments by the method given here would tend to be conservative (overestimate the moments). The limiting capacity of any components of the connection should, however, be recognized.

Table 3.1 Test-to-Predicted Ratios of the Flexible End Plate
Moment and Rotation at the Occurrence of Bottom Flange
Bearing

Test ¹ No.	TEST		KENNEDY-HAFEZ ANALYSIS				PROPOSED ANALYSIS			
	M_1^2 (in kips)	θ_1^3 (rads)	M_{pr} (in kips)	θ_{pr} (rads)	$\frac{M_1}{M_{pr}}$	$\frac{\theta_1}{\theta_{pr}}$	M_{pr} (in kips)	θ_{pr} (rads)	$\frac{M_1}{M_{pr}}$	$\frac{\theta_1}{\theta_{pr}}$
S5	791	0.0617	609	0.0536	1.30	1.15	729	0.0554	1.09	1.11
S6	190	0.0242	171	0.0164	1.11	1.48	181	0.0167	1.05	1.45
S7	361	0.0218	306	0.0197	1.18	1.11	333	0.0201	1.08	1.08
S8	625	0.0269	503	0.0247	1.24	1.09	580	0.0251	1.08	1.07
S9	971	0.0299	822	0.0319	1.18	0.937	996	0.0327	0.975	0.914
S10	329	0.0260	298	0.0363	1.10	0.716	326	0.0373	1.01	0.697
S11	678	0.0443	548	0.0450	1.24	0.984	619	0.0471	1.10	0.941
S12	425	0.0272	330	0.0594	1.29	1.19	1080	0.0623	1.20	1.16
S13	730	0.0316	586	0.0263	1.25	1.20	354	0.0234	1.14	1.16
S14	849	0.0408	665	0.0355	1.28	1.15	752	0.0272	1.13	1.12
S15	1410	0.0533	1090	0.0448	1.29	1.19	1230	0.0365	1.15	1.14
S16	258	0.0264	198	0.0211	1.30	1.25	217	0.0215	1.19	1.23
S17	393	0.0301	340	0.0271	1.16	1.11	398	0.0276	0.987	1.09
S18	752	0.0469	623	0.0354	1.21	1.32	677	0.0366	1.11	1.28
S19	1100	0.0480	995	0.0420	1.11	1.14	1110	0.0436	0.991	1.10
S20	443	0.0486	308	0.0376	1.44	1.29	360	0.0376	1.23	1.29
S25	163	0.0237	159	0.0276	1.03	0.859	173	0.0281	0.942	0.843
S26	207	0.0360	183	0.0728	1.13	0.495	216	0.0728	0.958	0.495
S27	76.4	0.0386	68.1	0.0442	1.12	0.873	74.4	0.0452	1.03	0.854

Table 3.1 Test-to-Predicted Ratios of the Flexible End Plate
Moment and Rotation at the Occurrence of Bottom Flange
Bearing

Test ¹ No.	TEST		KENNEDY-HAFEZ ANALYSIS				PROPOSED ANALYSIS			
	M _t ² (in kips)	θ _t ³ (rads)	M _{pr} (in kips)	θ _{pr} (rads)	M _t M _{pr}	θ _t θ _{pr}	M _{pr} (in kips)	θ _{pr} (rads)	M _t M _{pr}	θ _t θ _{pr}
H1	1260	0.0272	1140	0.0303	1.11	0.898	1290	0.0312	0.977	0.872
H2	940	0.0296	729	0.0229	1.29	1.27	844	0.0233	1.11	1.27
H3	900	0.0395	818	0.0343	1.10	1.15	933	0.0355	0.965	1.11
H4	255	0.0321	194	0.0166	1.31	1.93	206	0.0169	1.24	1.90
H5	365	0.0290	333	0.0238	1.10	1.22	355	0.0241	1.03	1.20
H6	490	0.0335	521	0.0296	0.941	1.13	557	0.0303	0.880	1.11
H7	1780	0.0394	2075	0.0573	0.858	0.688	2270	0.0618	0.784	0.638
H8 ⁴	-	-	1570	0.0448	-	-	1710	0.0490	-	-
			Mean Value							
			Standard Dev.							
			Coef. of Var.							
			1.18				1.06			
			0.126				0.109			
			0.107				0.103			

1 S refers to tests performed by Sommers (1969) and H refers to tests performed by Hafez (1982).

2 The lesser moment observed when the flange did not contact the support simultaneously on both sides of the web.

3 The average of the two rotations observed when the flange did not contact the support simultaneously on both sides of the web.

4 In this test, the connection failed before the beam came into contact with the support.

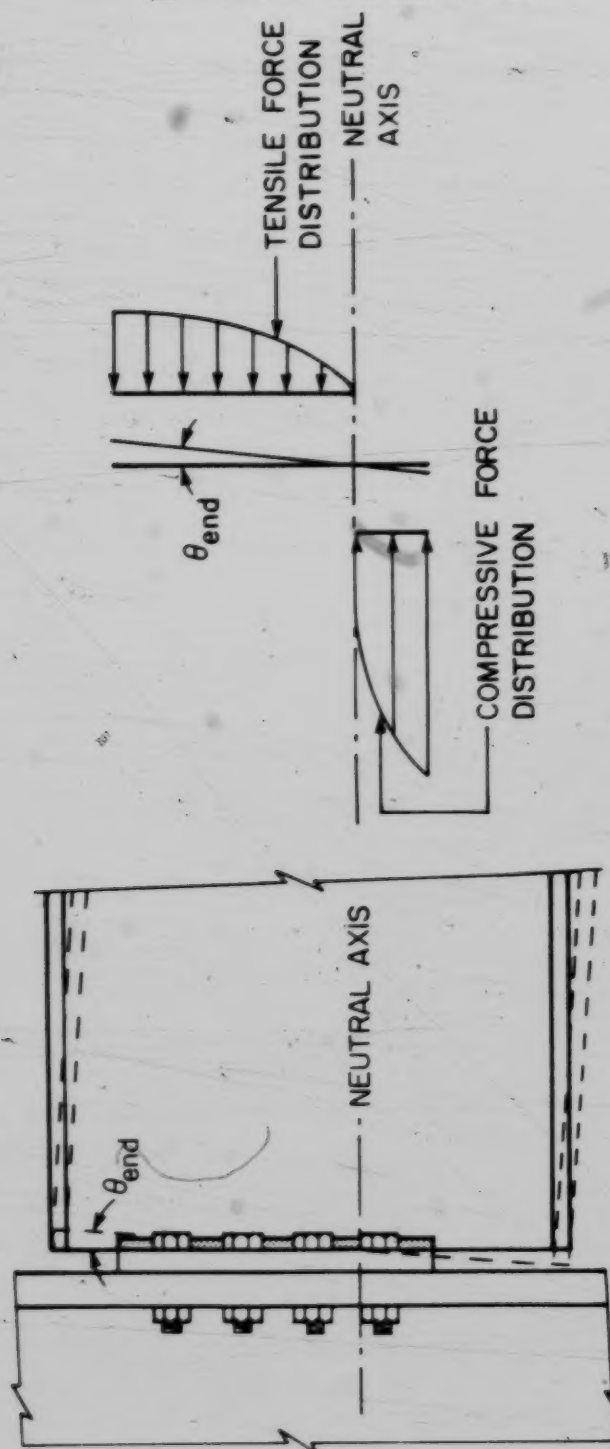


Figure 3.1 Flexural Force Distribution at Flexible End Plate Connection

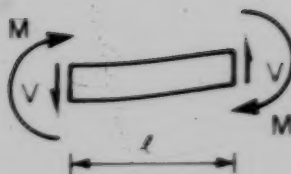
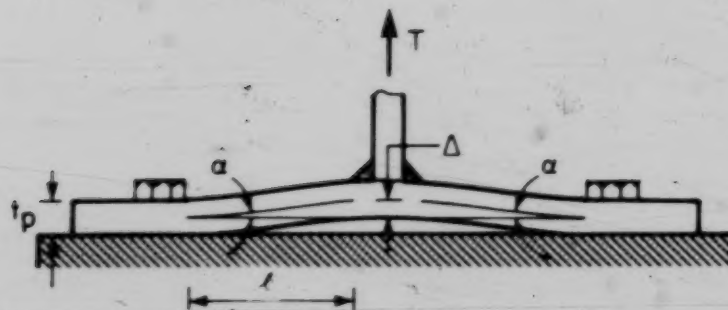


Figure 3.2 Flexible End Plate Connection, Elastic Tension
Region Response

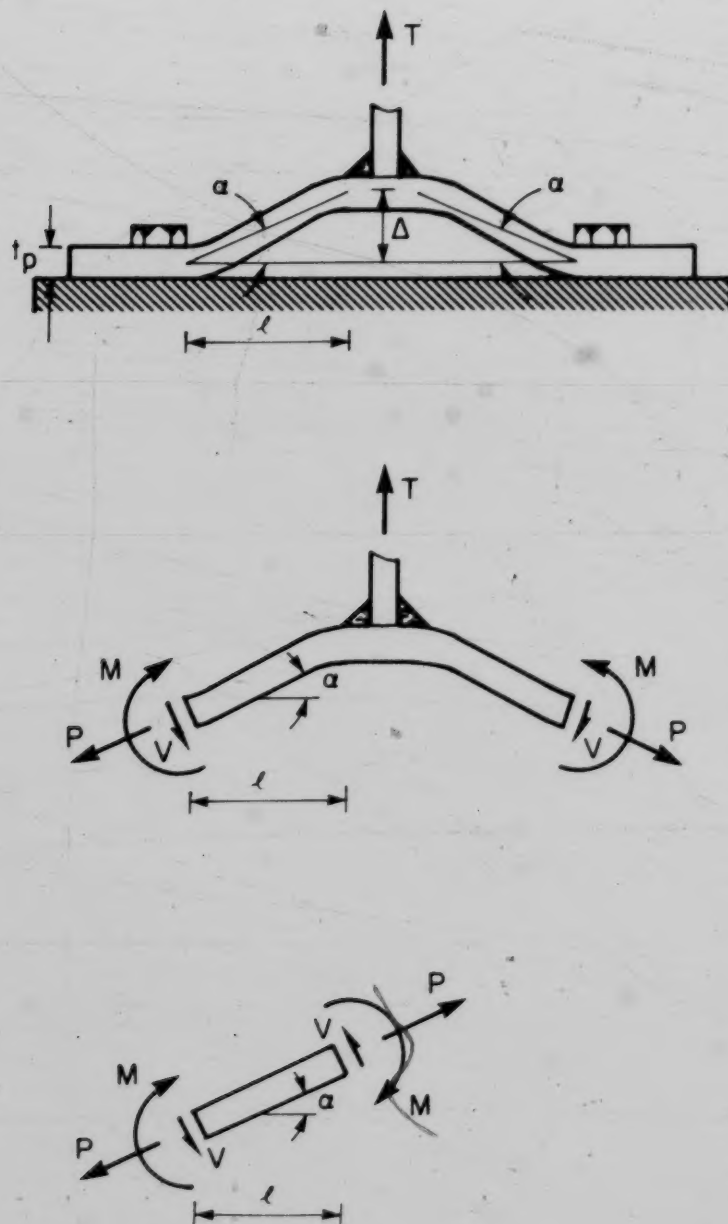


Figure 3.3 Flexible End Plate Connection, Inelastic Tension
Region Response

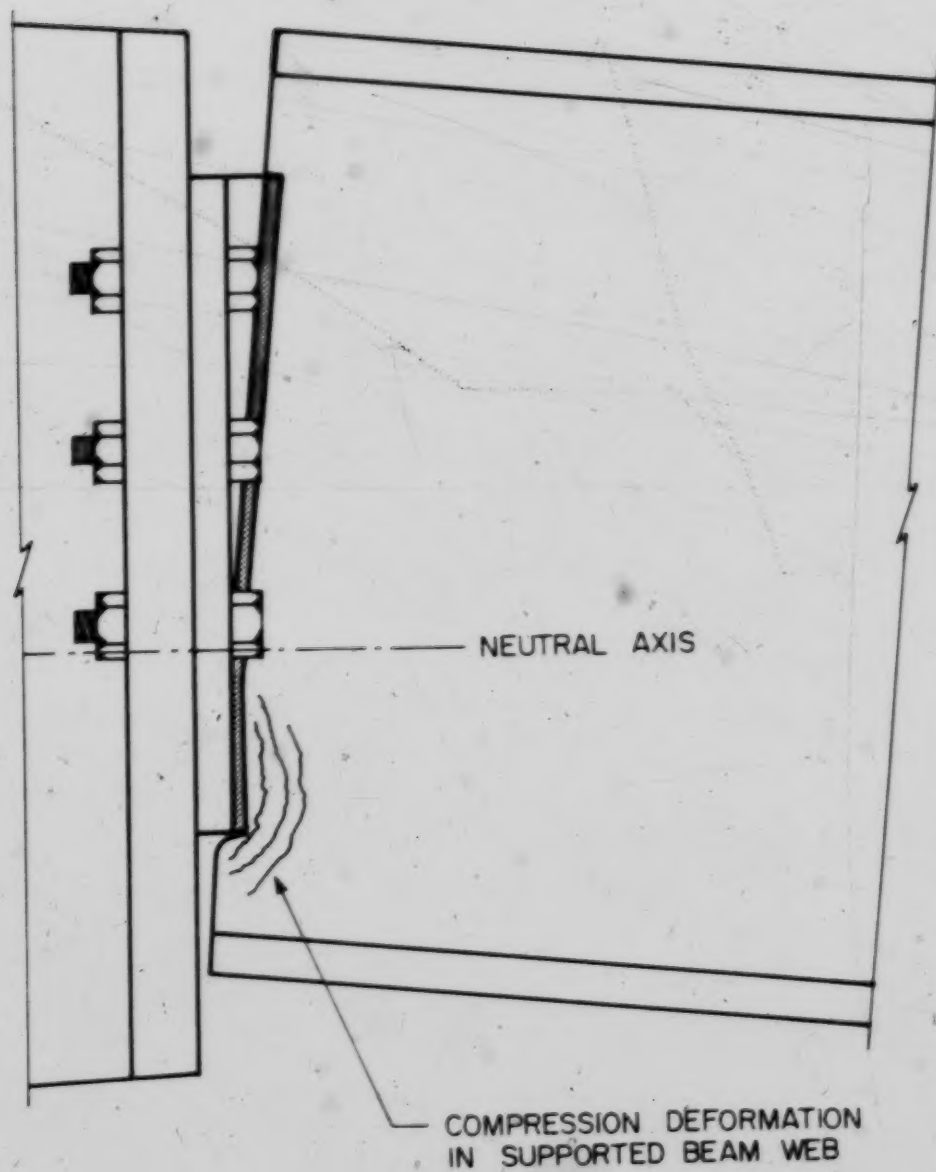
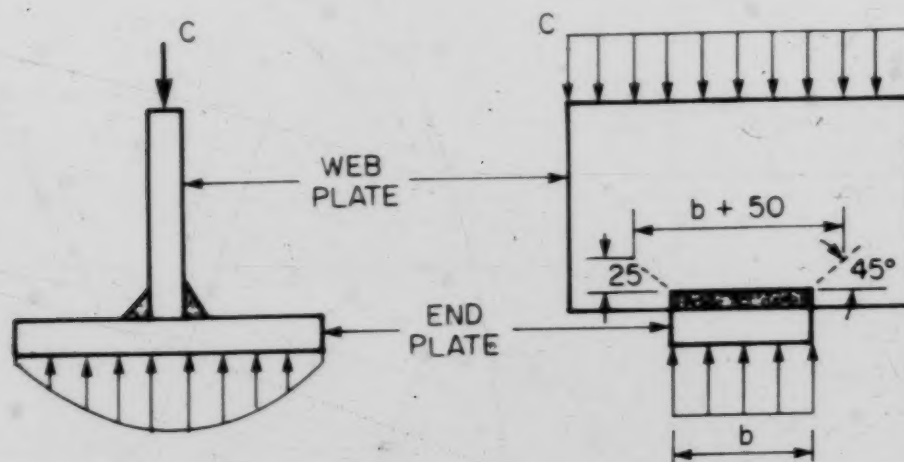
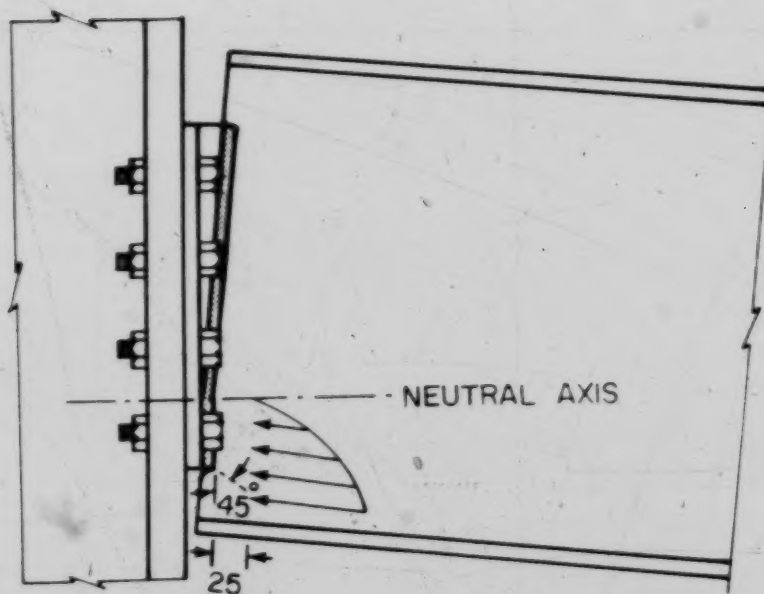


Figure 3.4 Behaviour of Compressed Region of Flexible End
Plate Connection



a) END PLATE COMPRESSION LOAD DEFORMATION TEST SPECIMENS



b) SPREAD OUT OF COMPRESSIVE STRESSES IN WEB OF SUPPORTED MEMBER

Figure 3.5 Stress Distribution in Compressed Region of Flexible End Plate Connection

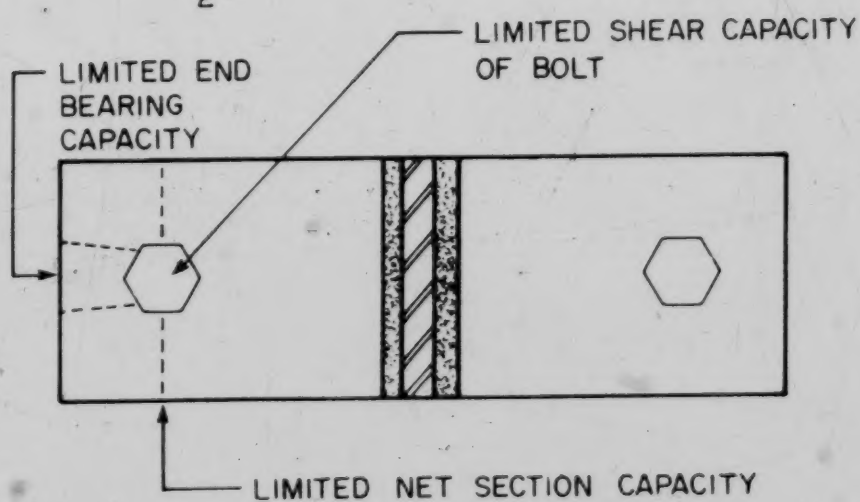
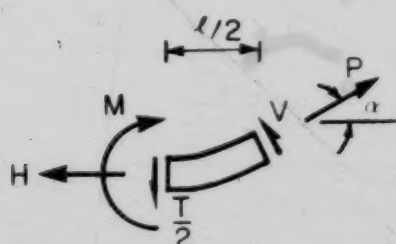
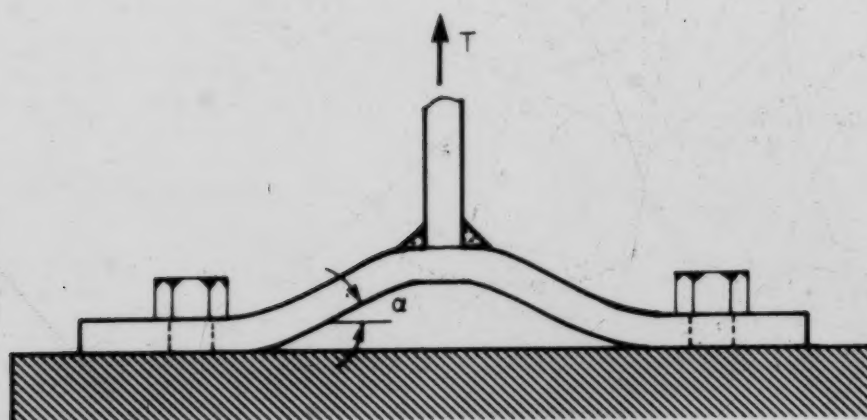


Figure 3.6 Factors Limiting the Development of Membrane Forces in Tension Region of Flexible End Plate

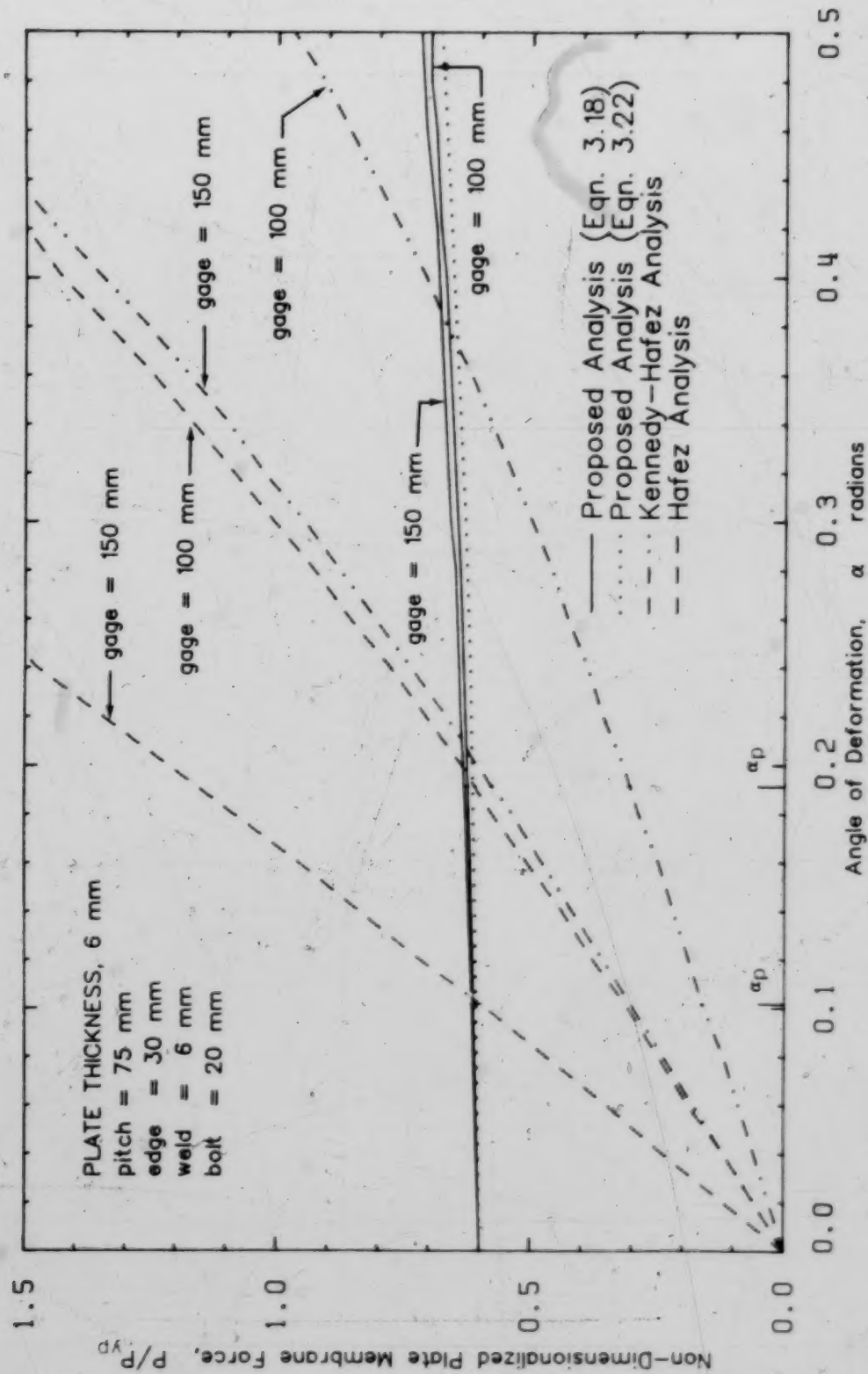


Figure 3.7 Maximum Membrane Force in a Flexible End Plate vs. Angle of Deformation

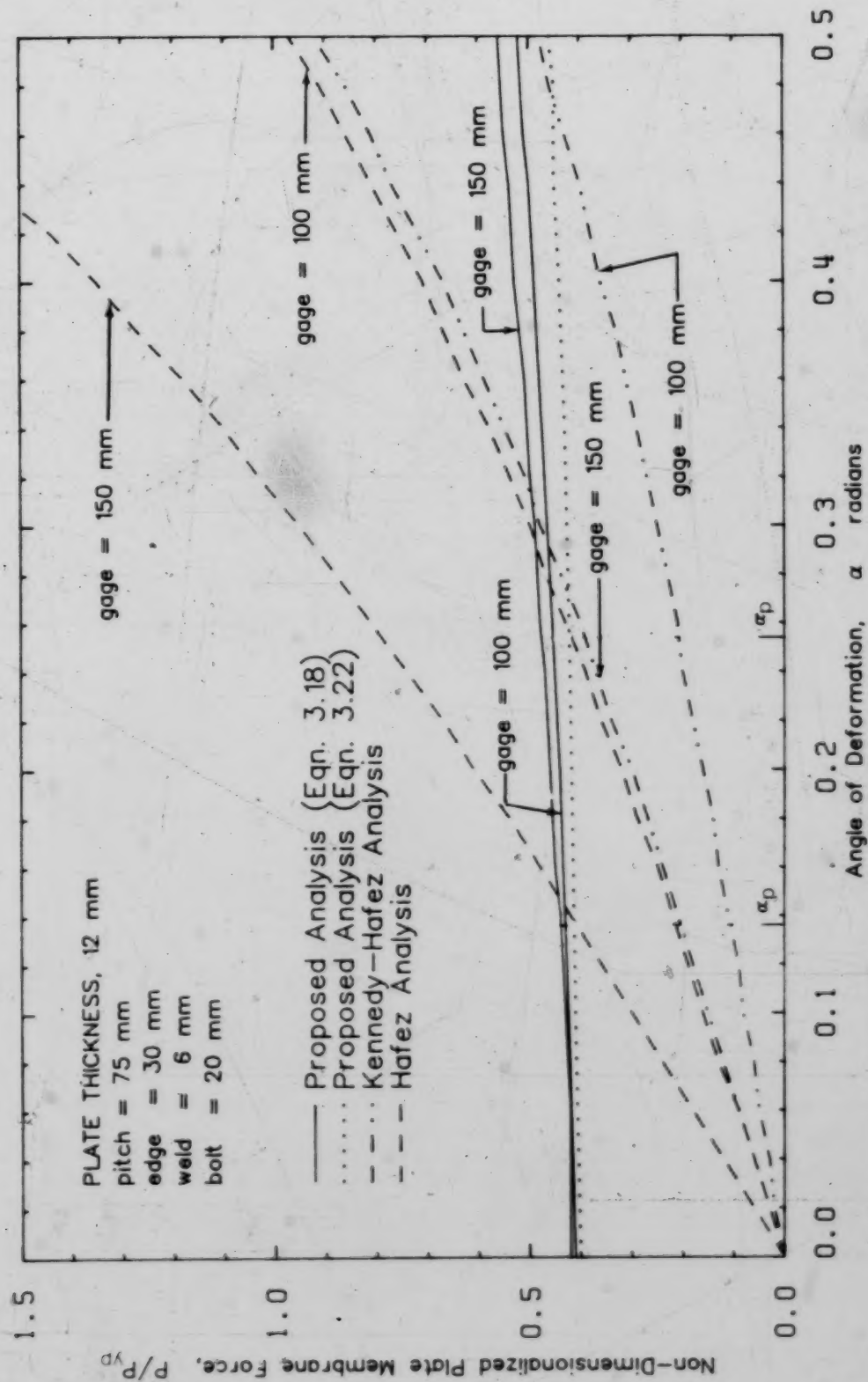


Figure 3.8 Maximum Membrane Force in a Flexible End Plate vs. Angle of Deformation

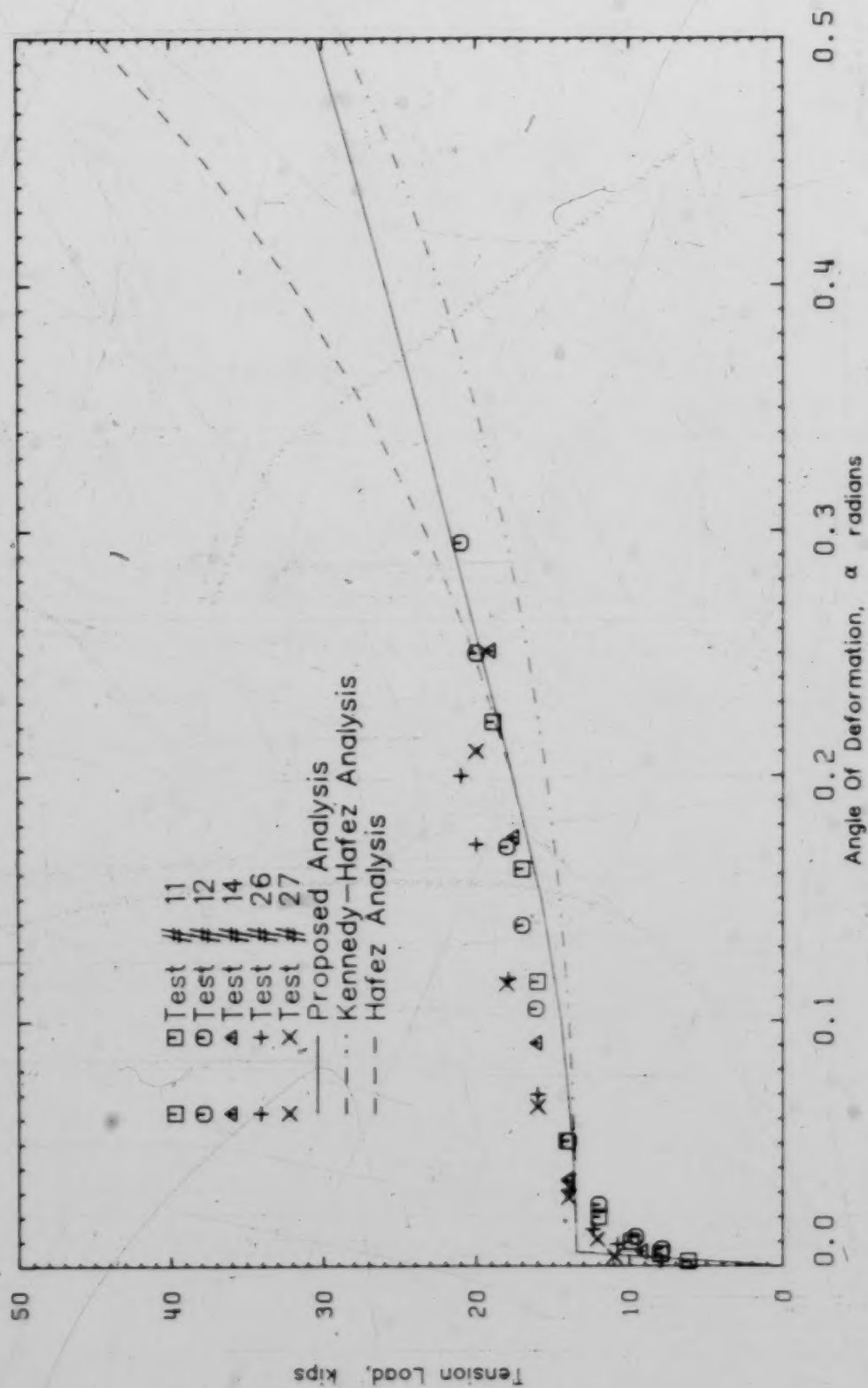


Figure 3.9 Comparison of Analytical Load-Deformation Relationships with Hafez (1982) T-Section Tests, 1/4" Plate and 4" Gage

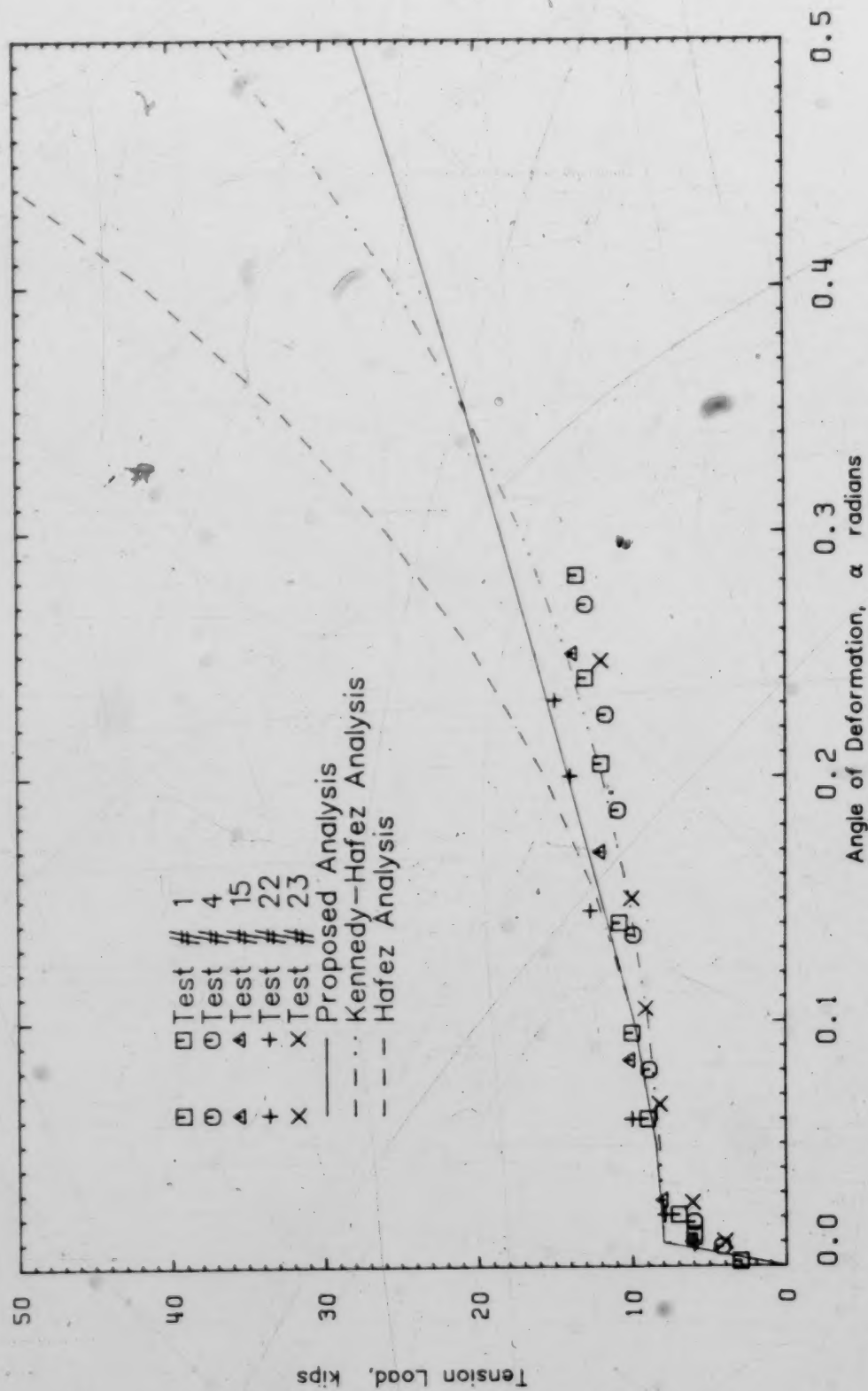


Figure 3.10 Comparison of Analytical Load-Deformation Relationships with Hafez (1982) T-Section Tests, 1/4" Plate and 5 1/2" Gage

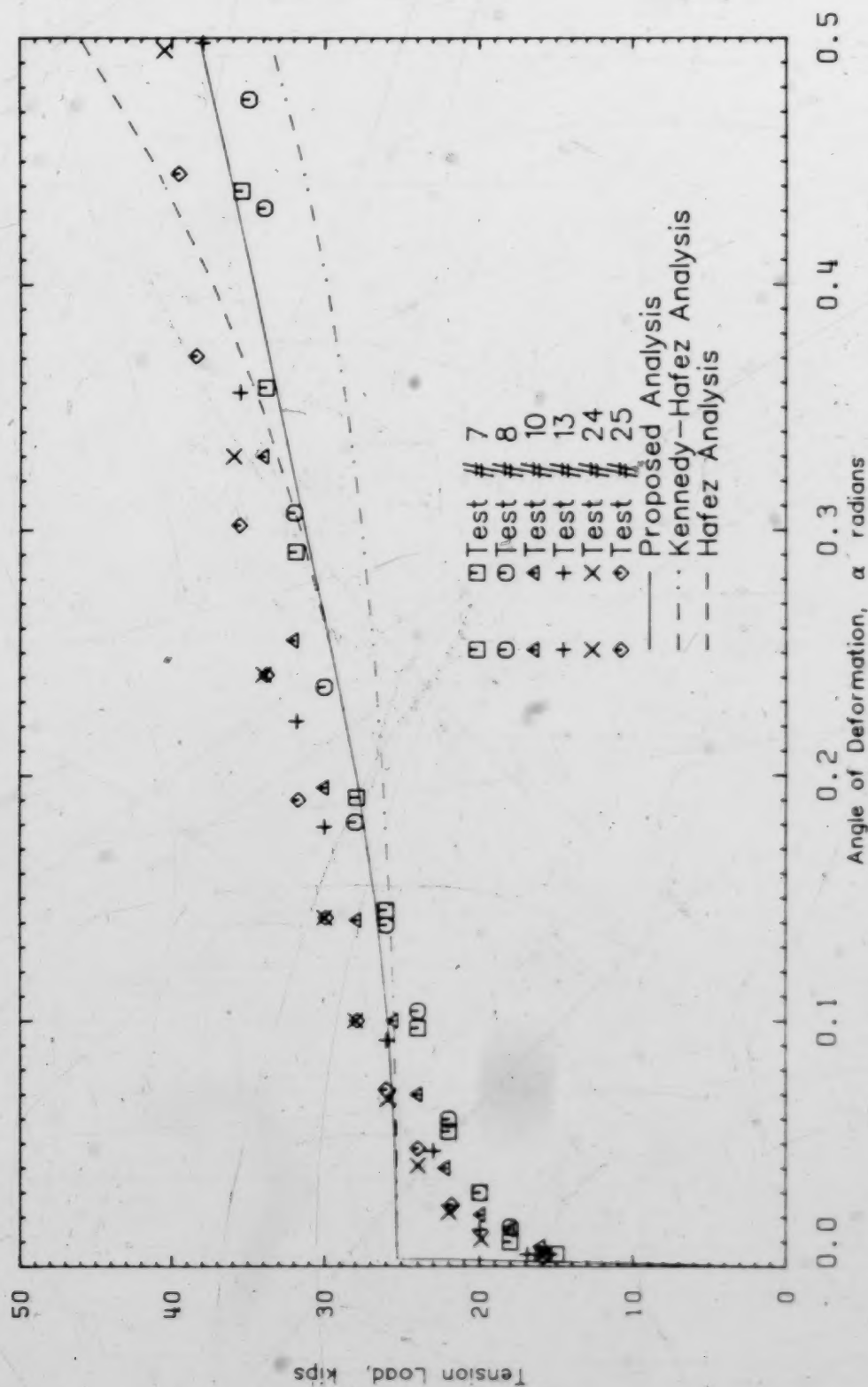


Figure 3.11 Comparison of Analytical Load-Deformation Relationships with Hafez (1982) T-Section Tests, 3/8" Plate and 4" Girders

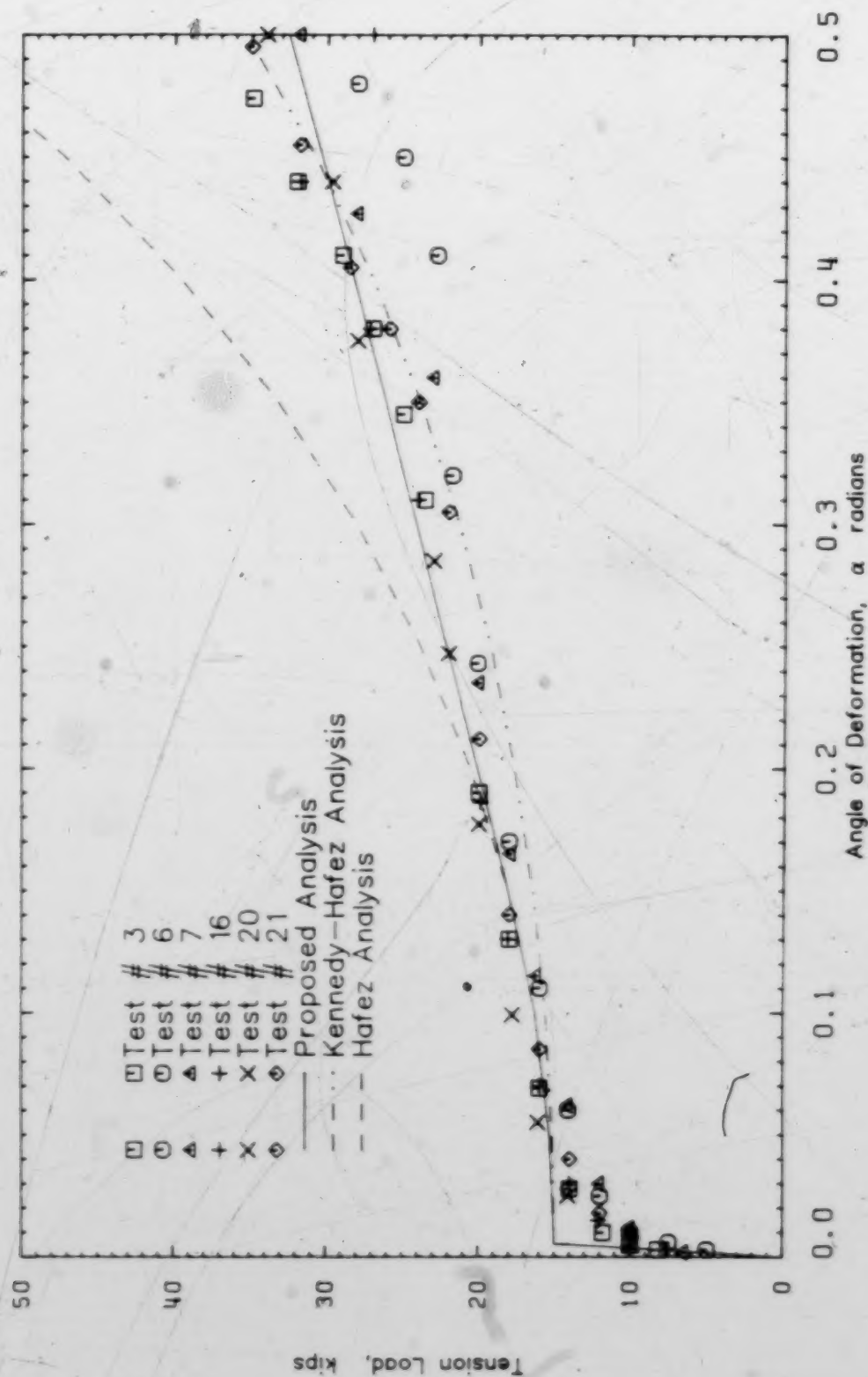


Figure 3.12 Comparison of Analytical Load-Deformation Relationships with Hafez (1982) T-Section Tests, $3/8$ " Plate and $5 1/2$ " Gage

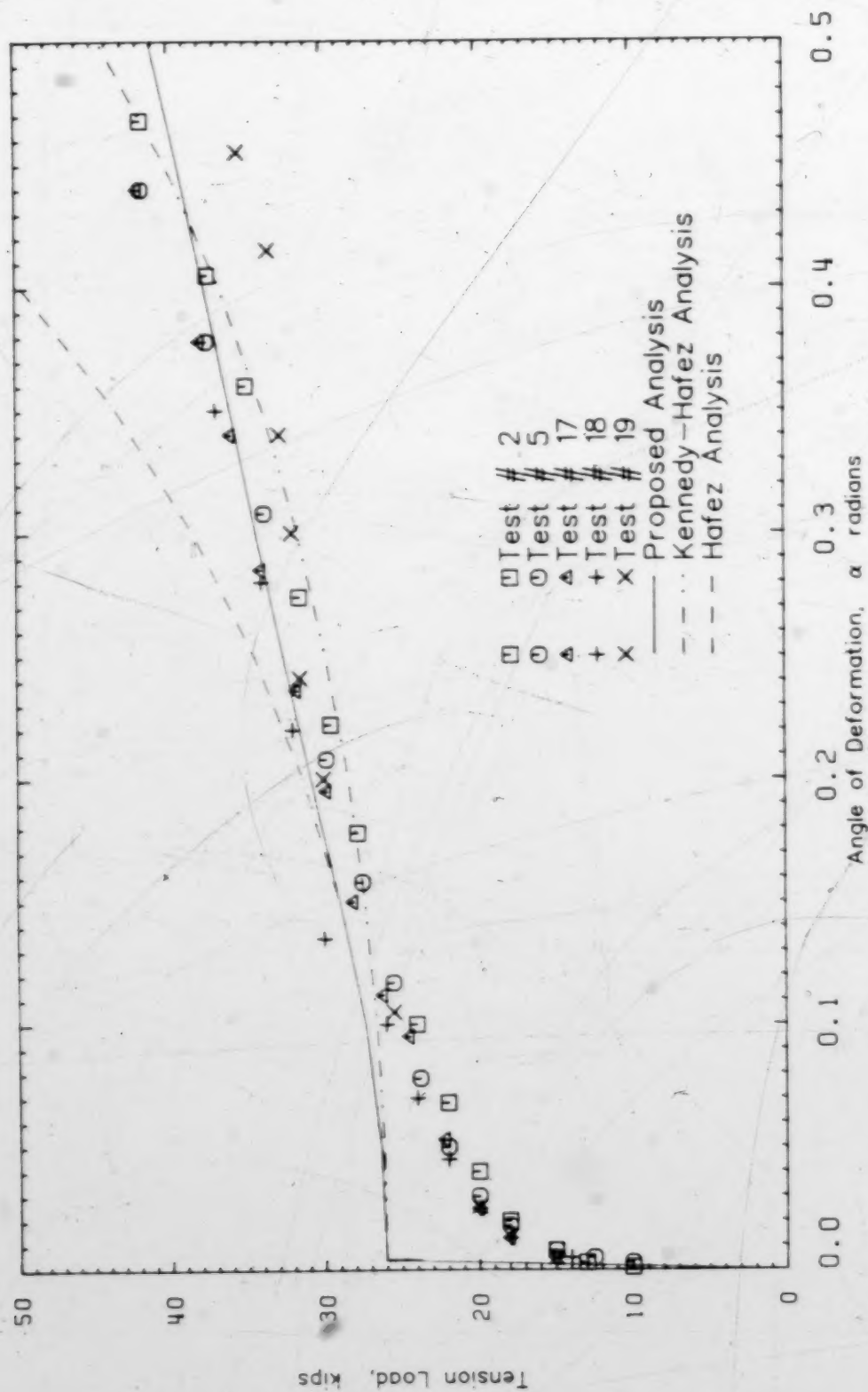
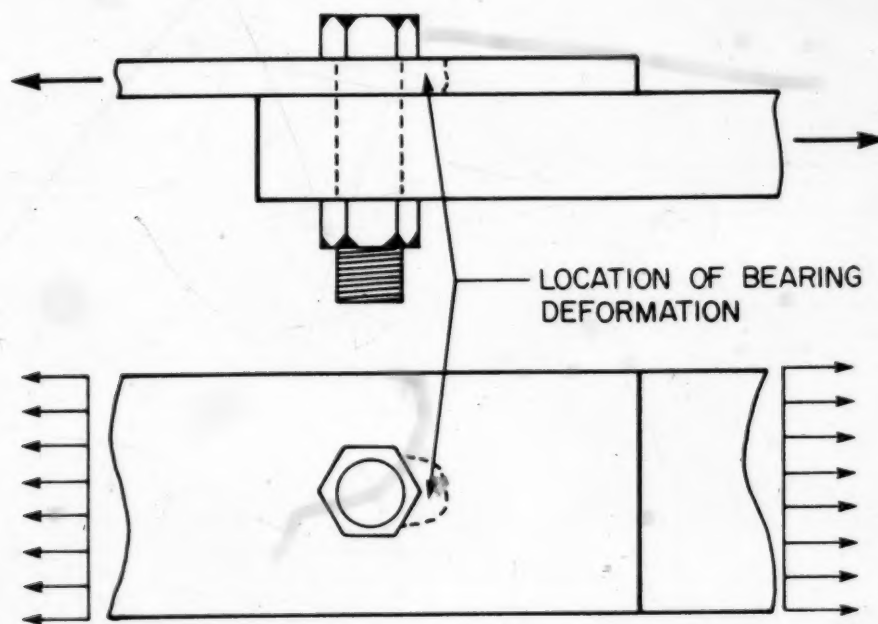
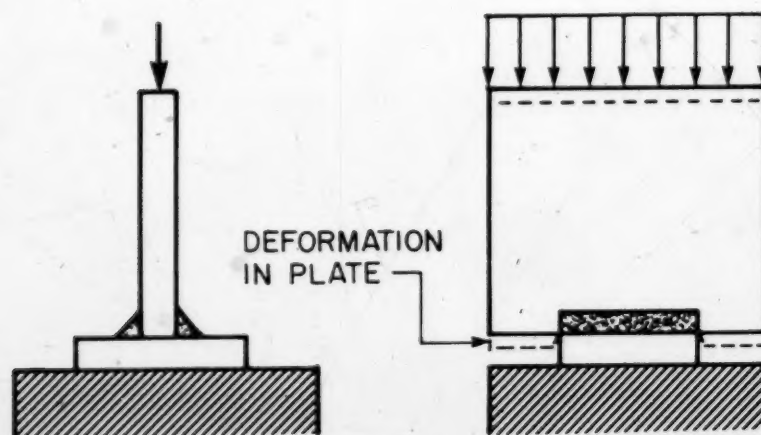


Figure 3.13 Comparison of Analytical Load-Deformation Relationships with Hafez (1982) T-Section Tests, 1/2" Plate and 5 1/2" Gage



a) BOLT BEARING TEST SPECIMEN



b) END PLATE TEST SPECIMEN

Figure 3.14 Bearing Deformation Specimens

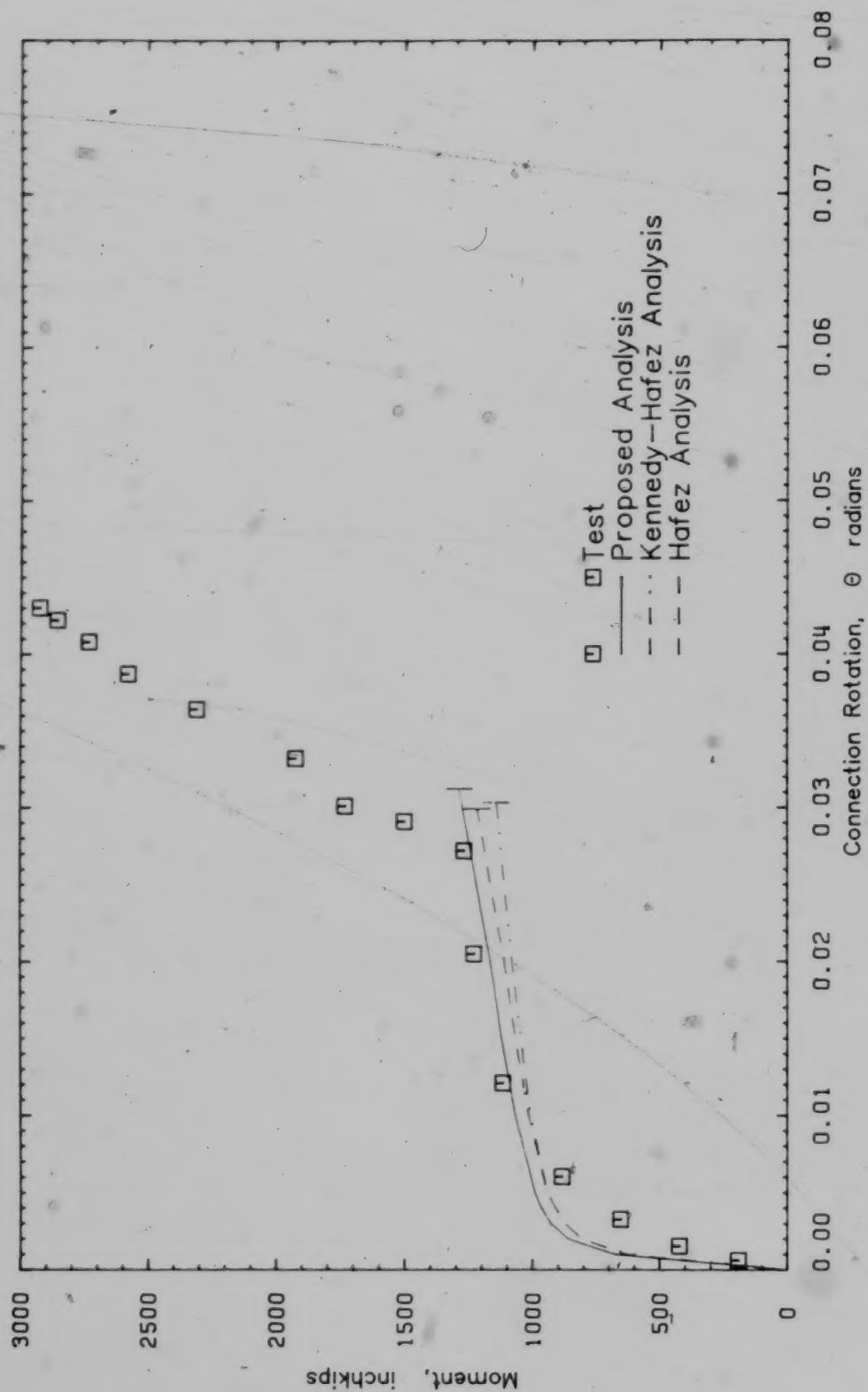


Figure 3.15 Comparison Of Analytical Moment-Rotation Relationships with Hafez (1982) End Plate Test # 1

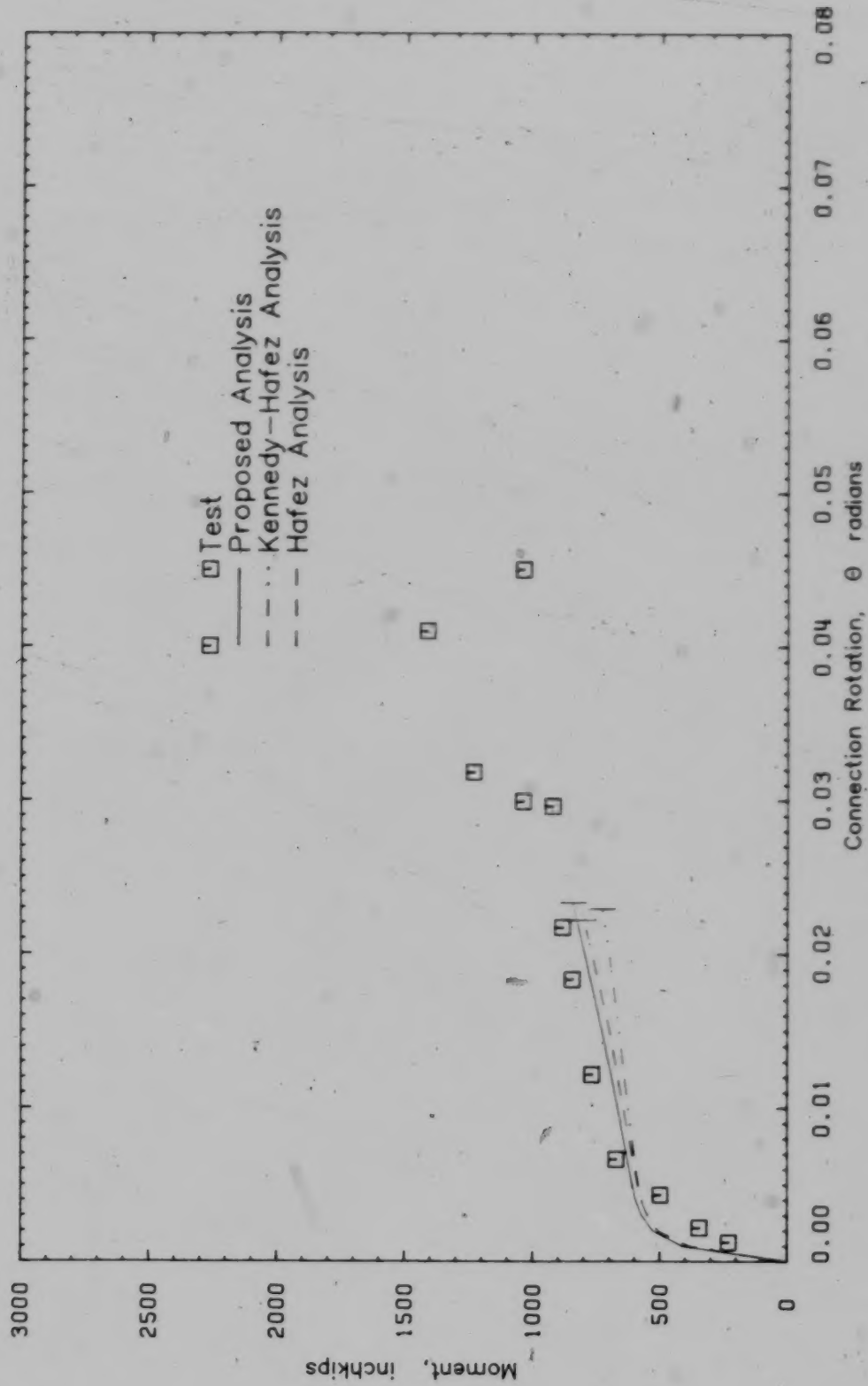


Figure 3.16 Comparison of Analytical Moment-Rotation Relationships with Hafez (1982) End Plate Test # 2

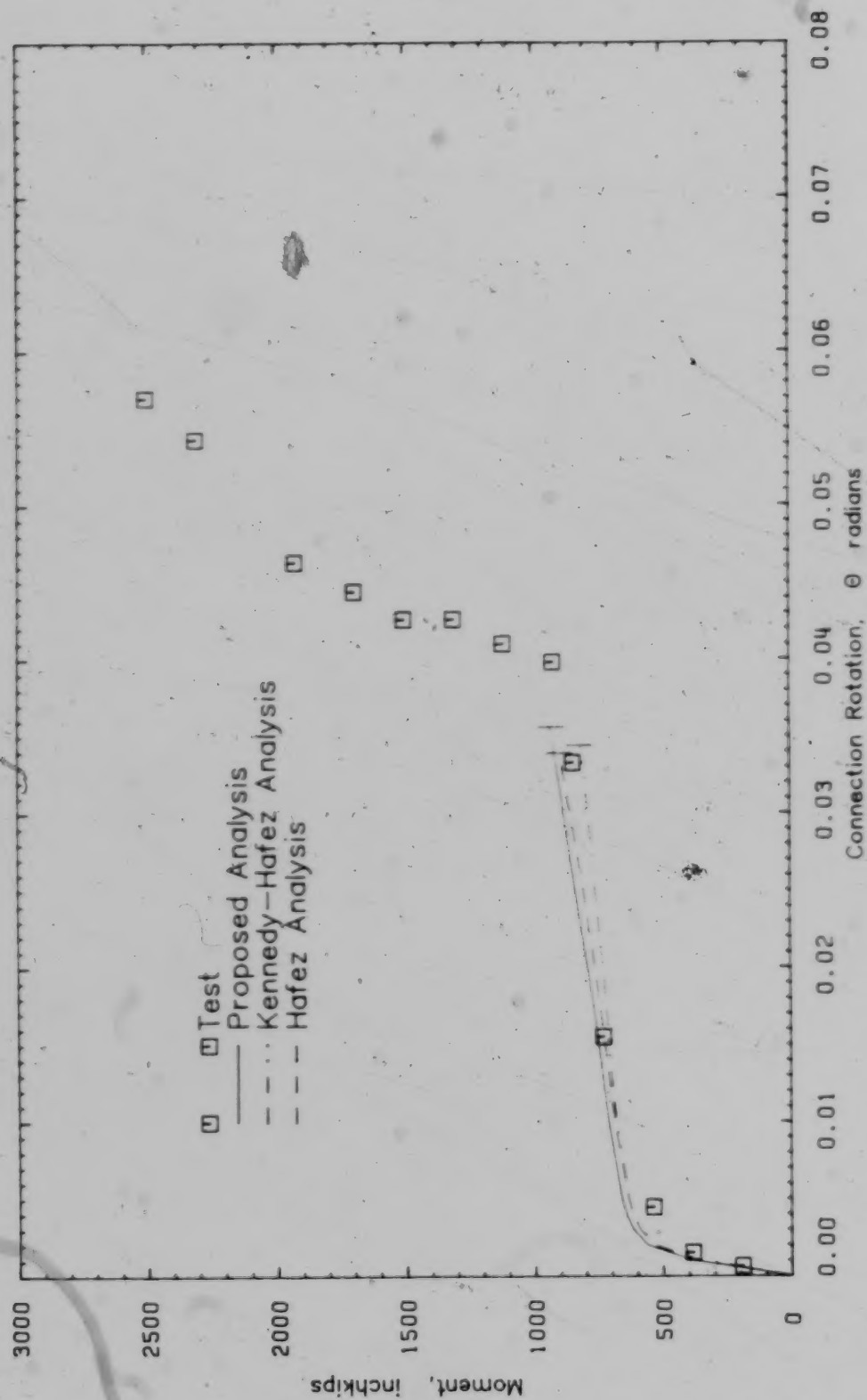


Figure 3.17 Comparison of Analytical Moment-Rotation Relationships with Hafez (1982) End Plate Test # 3

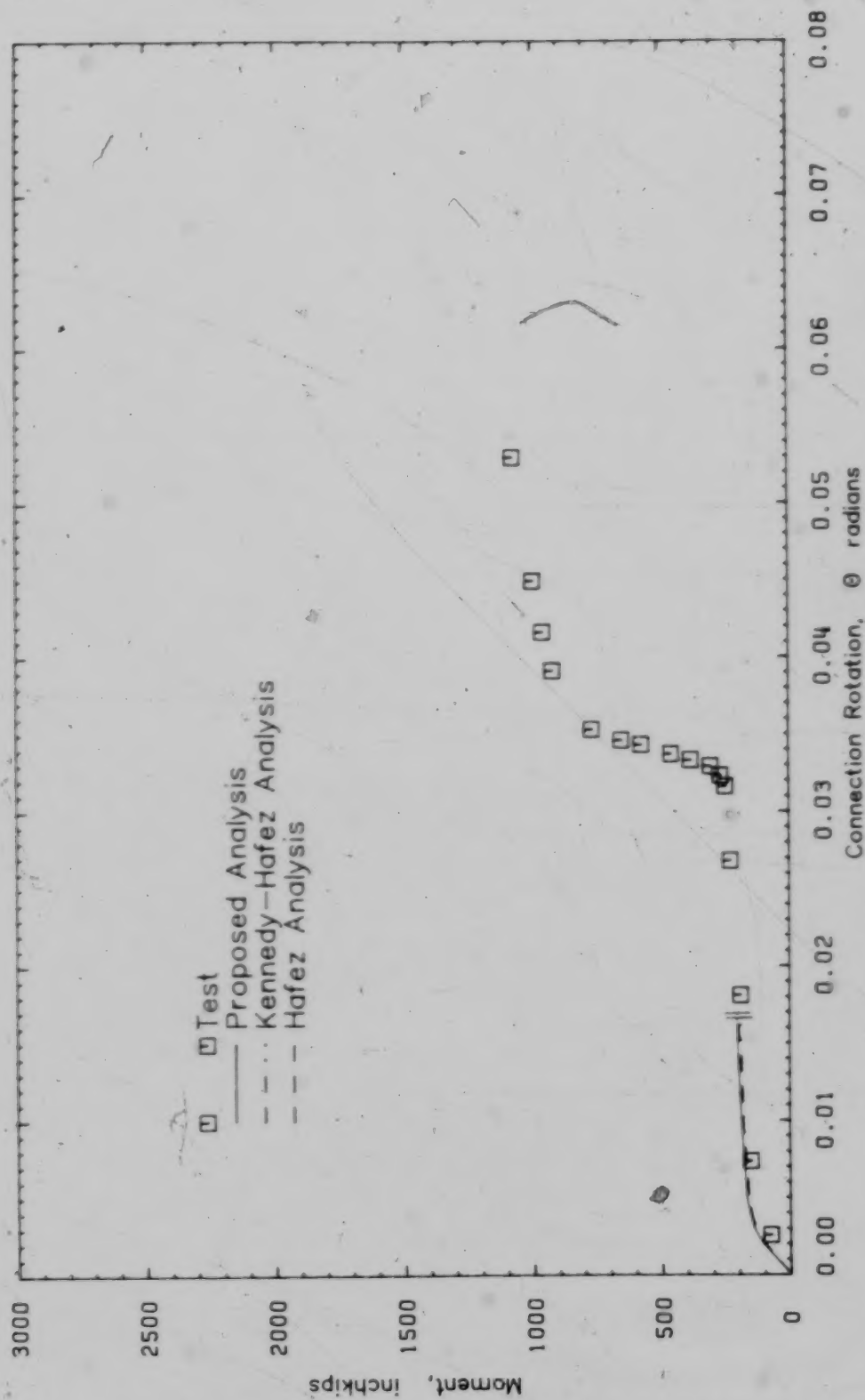


Figure 3.18 Comparison of Analytical Moment-Rotation Relationships with Hafez (1982) End Plate Test # 4

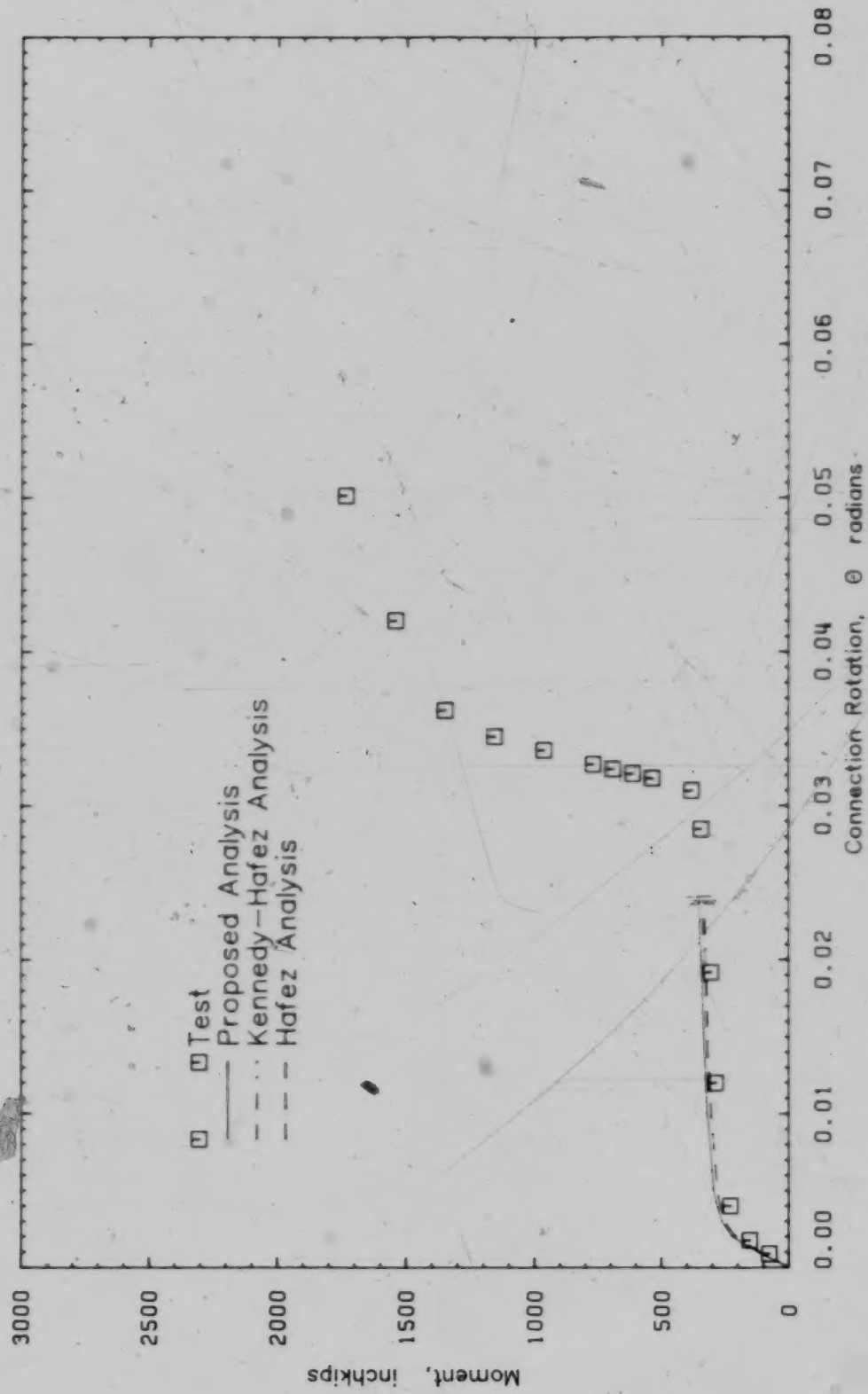


Figure 3.19 Comparison of Analytical Moment-Rotation Relationships with Hafez (1982) End Plate Test # 5

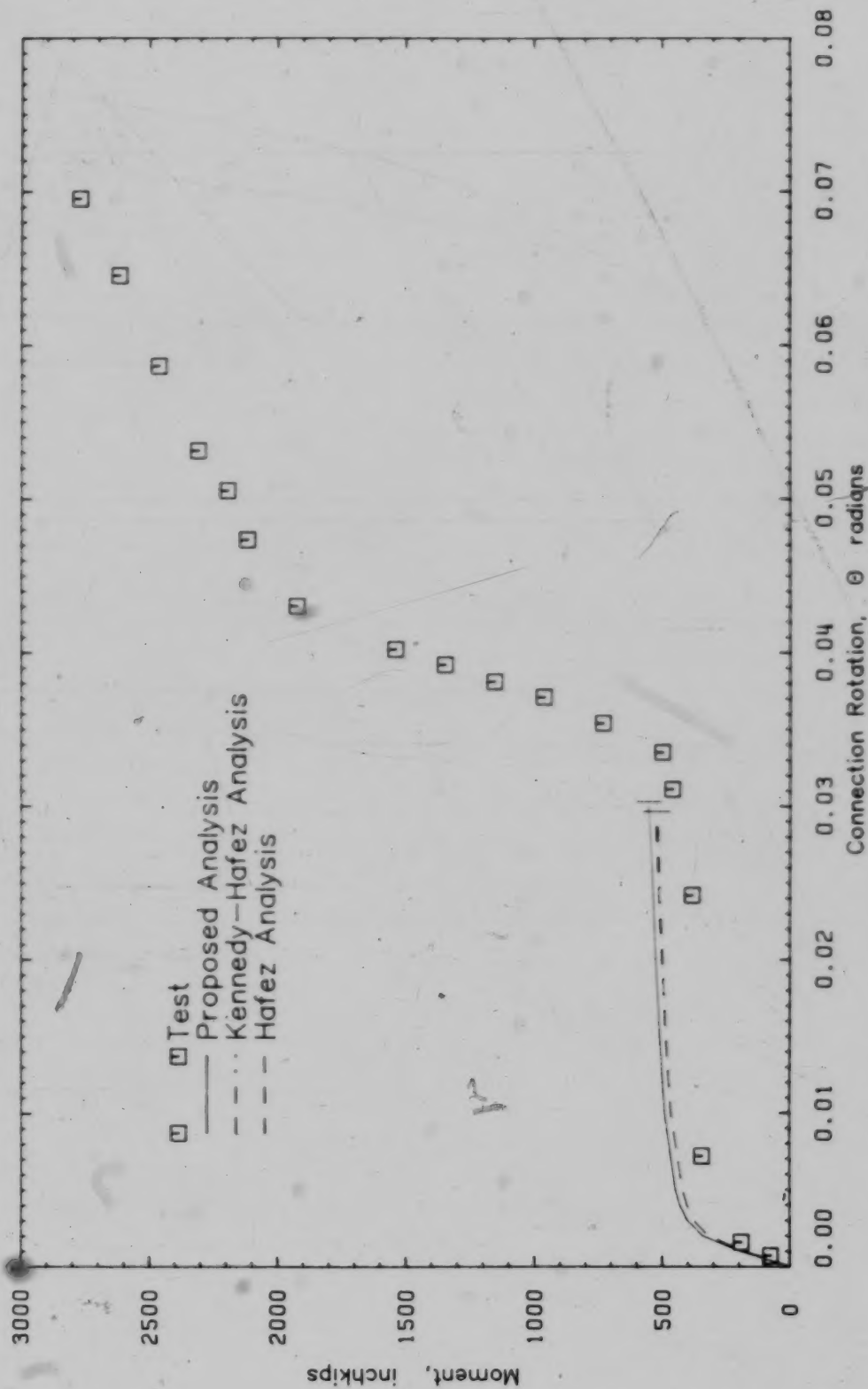


Figure 3.20 Comparison of Analytical Moment-Rotation Relationships with Hafez (1982) End Plate Test # 6

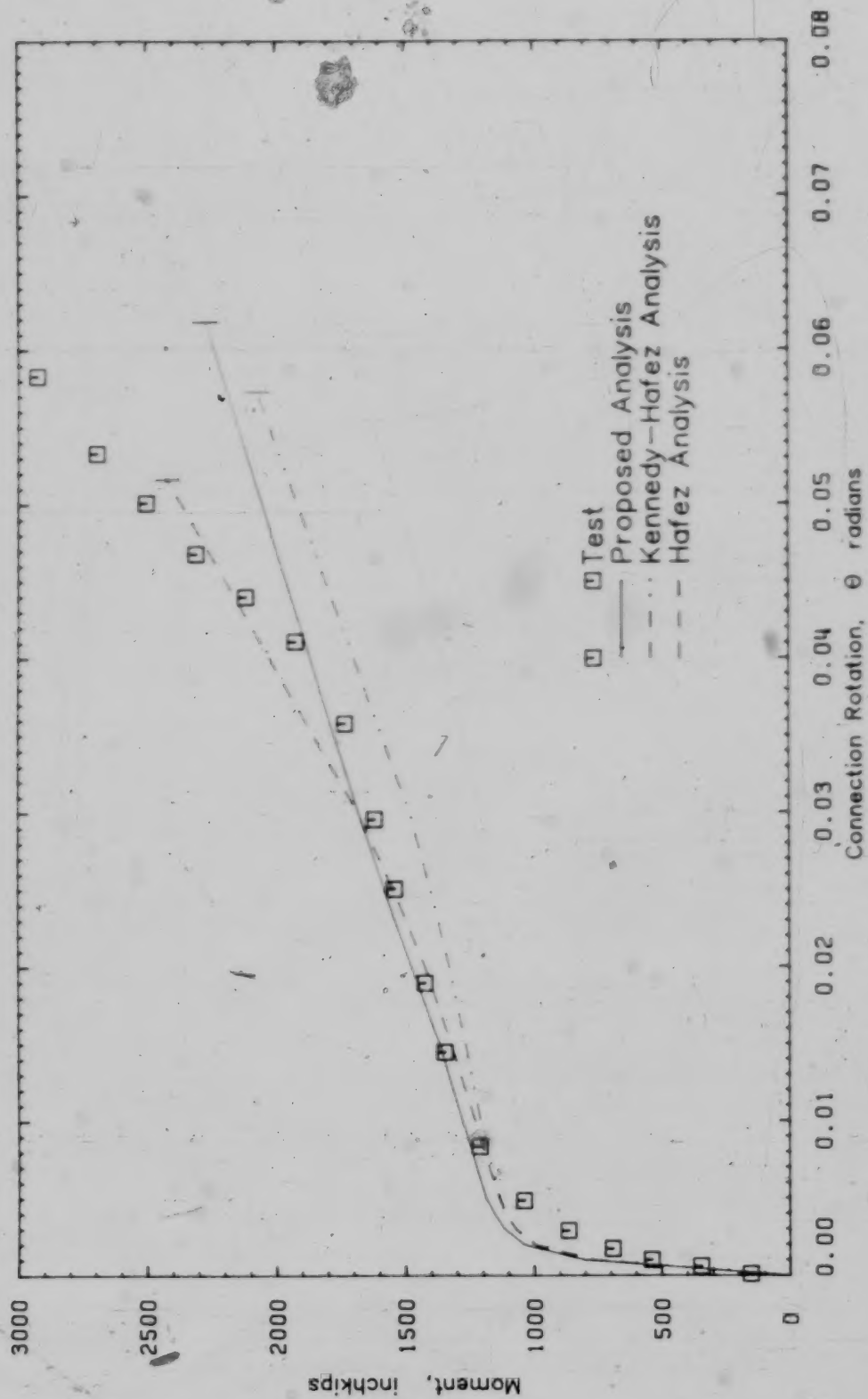


Figure 3.21 Comparison of Analytical Moment-Rotation Relationships with Hafez (1982) End Plate Test # 7

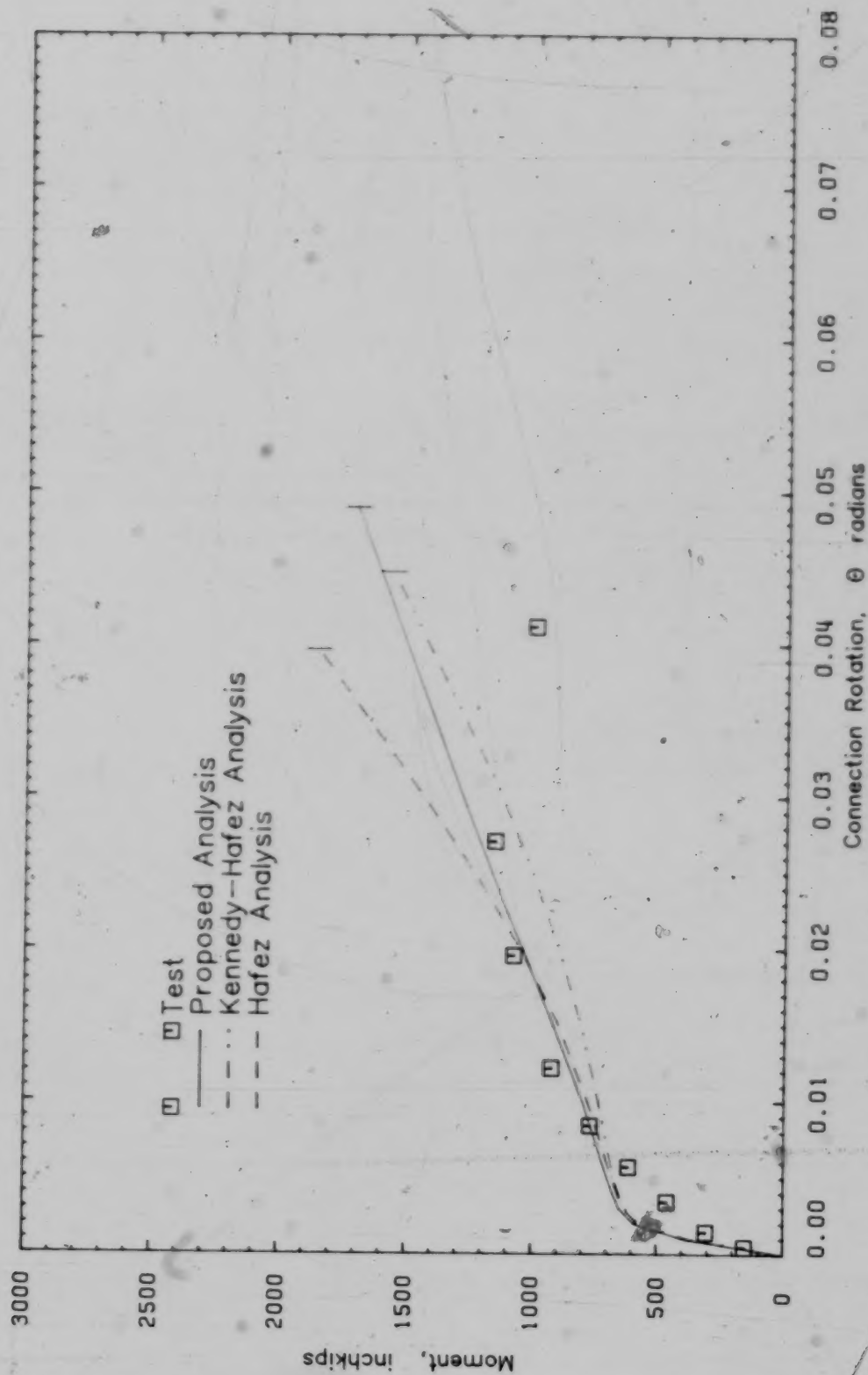


Figure 3.22 Comparison of Analytical Moment-Rotation Relationships with Hafez (1982) End Plate Test # 8

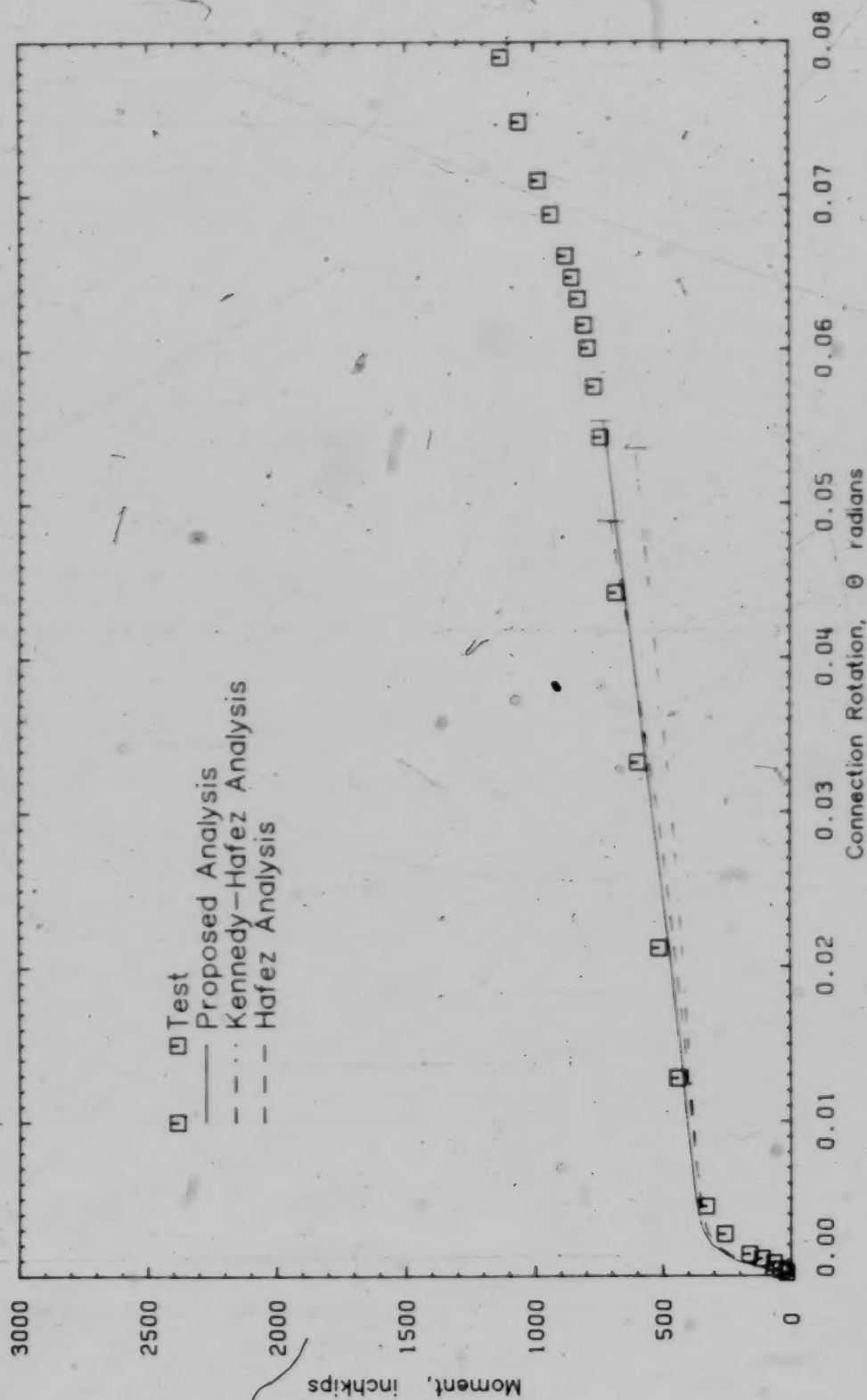


Figure 3.23 Comparison of Analytical Moment-Rotation Relationships
with Sommer (1969) End Plate Test # 5

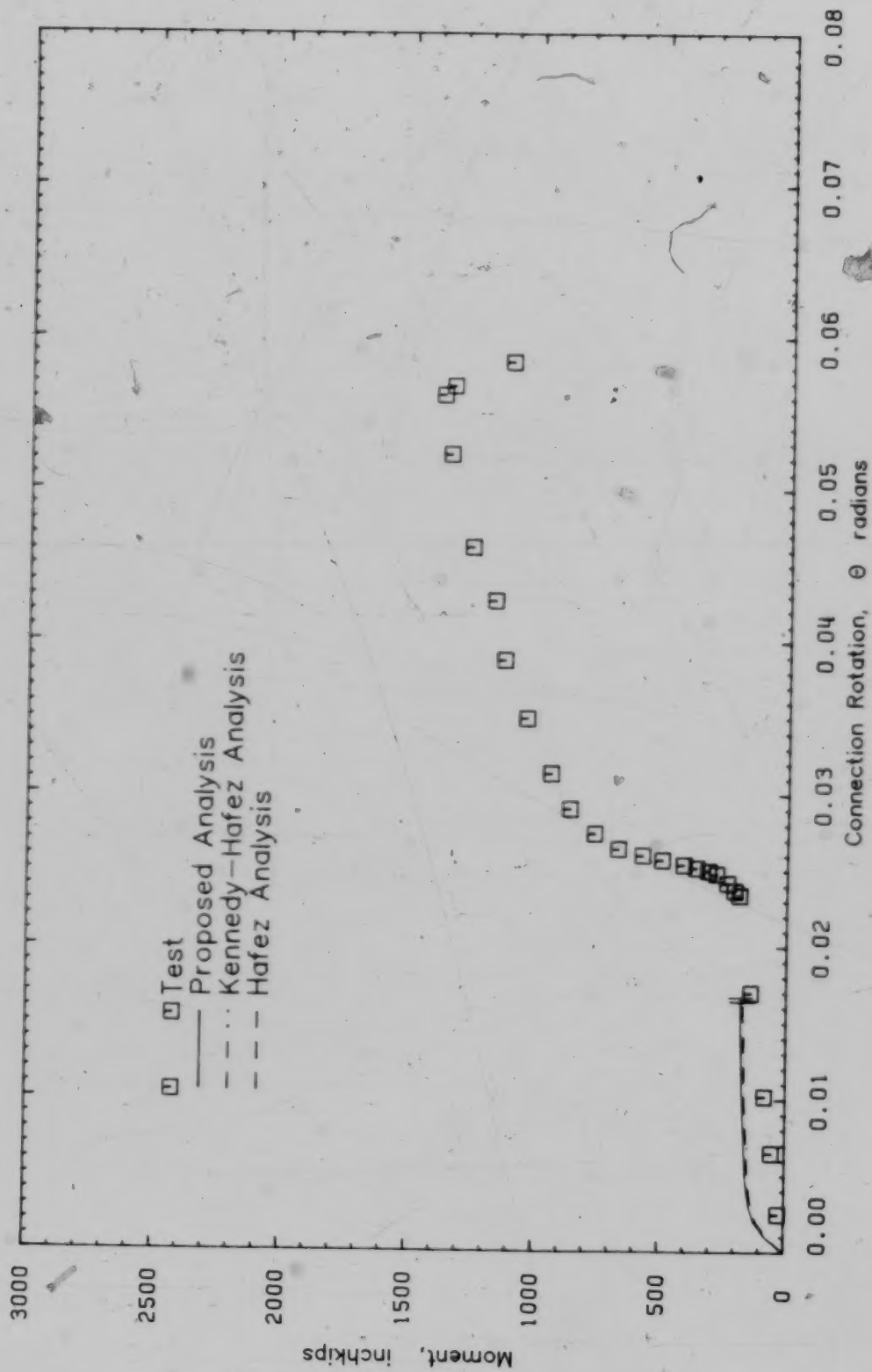


Figure 3.24 Comparison Of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 6

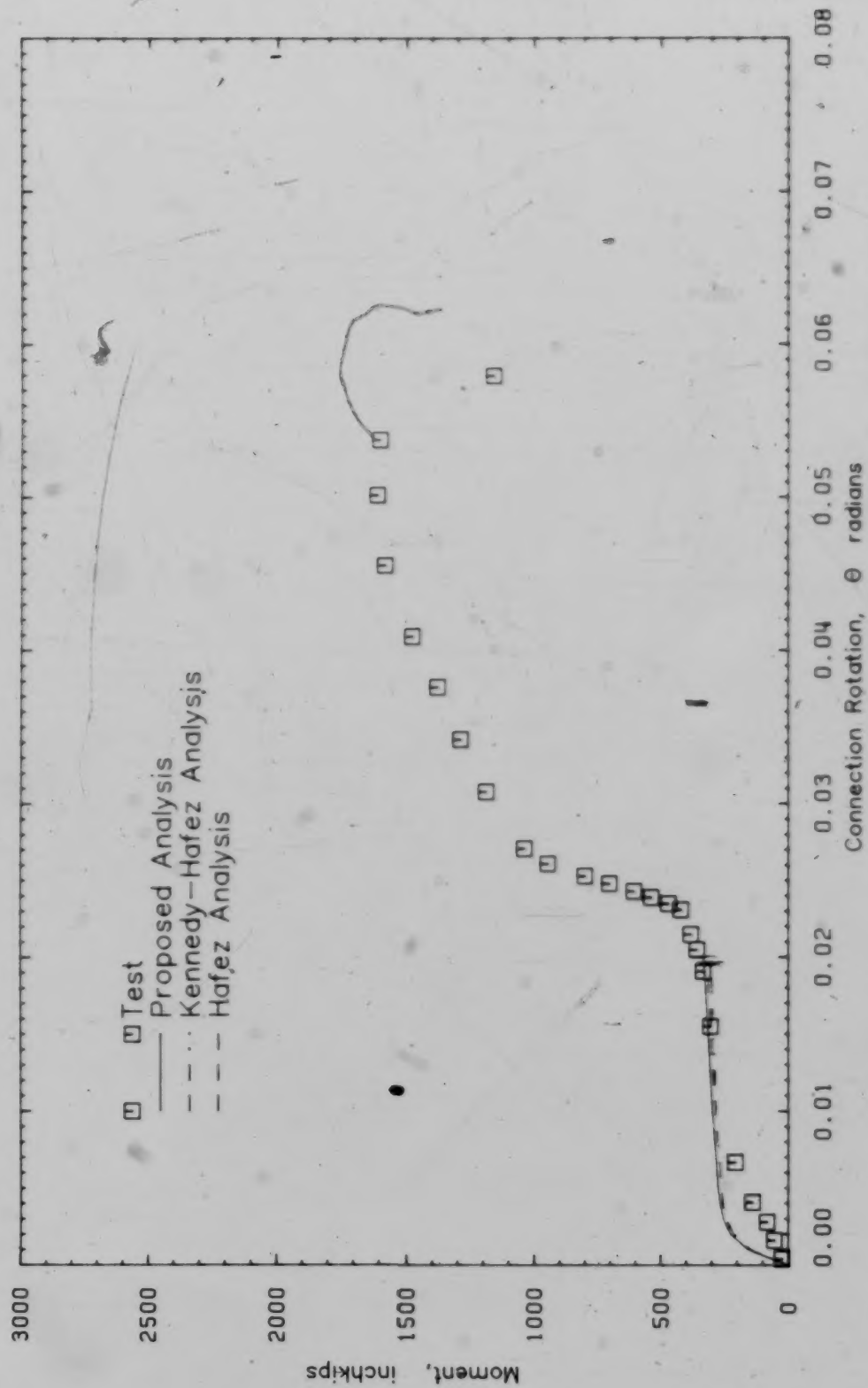


Figure 3.25 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 7

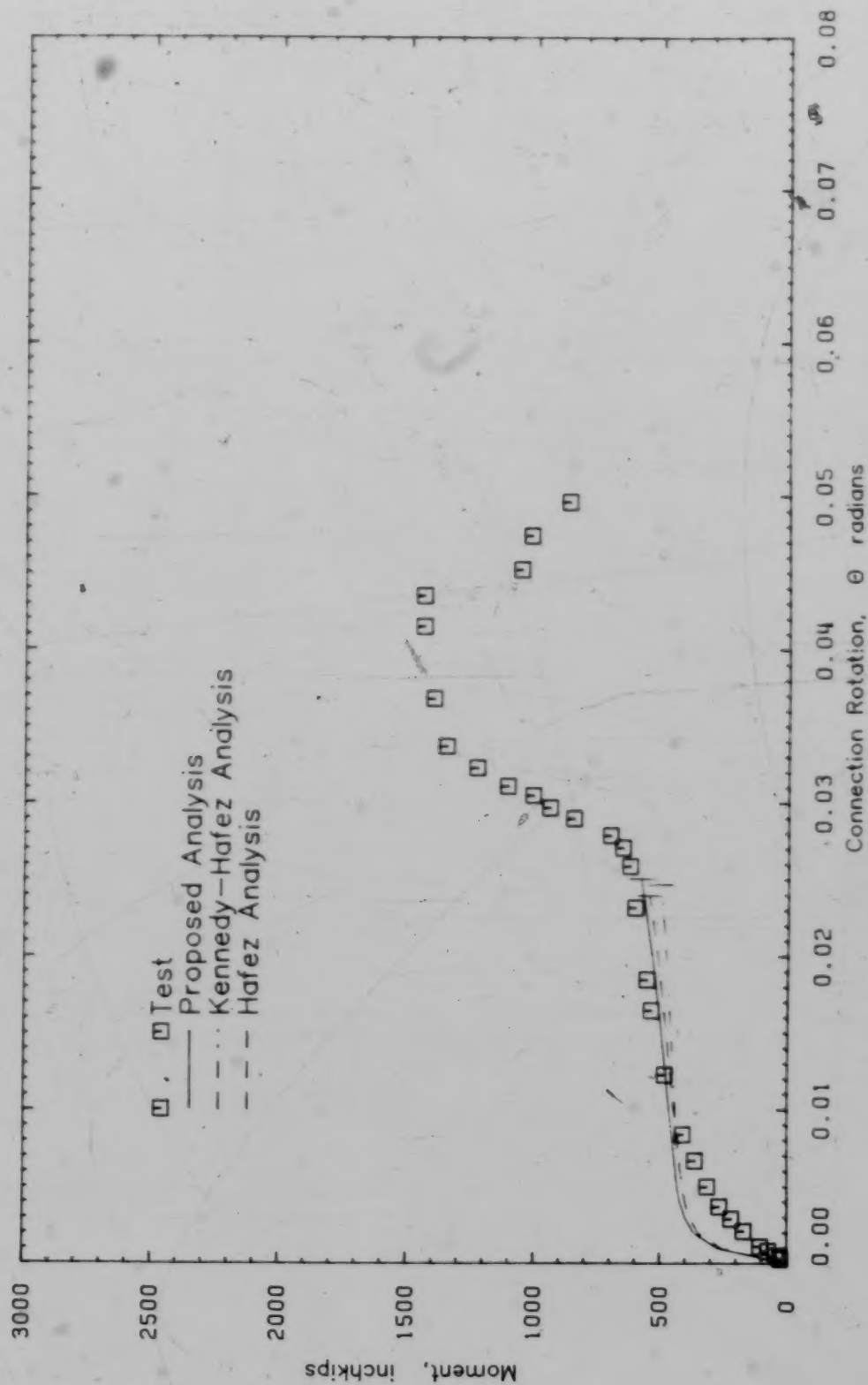


Figure 3.26 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 8

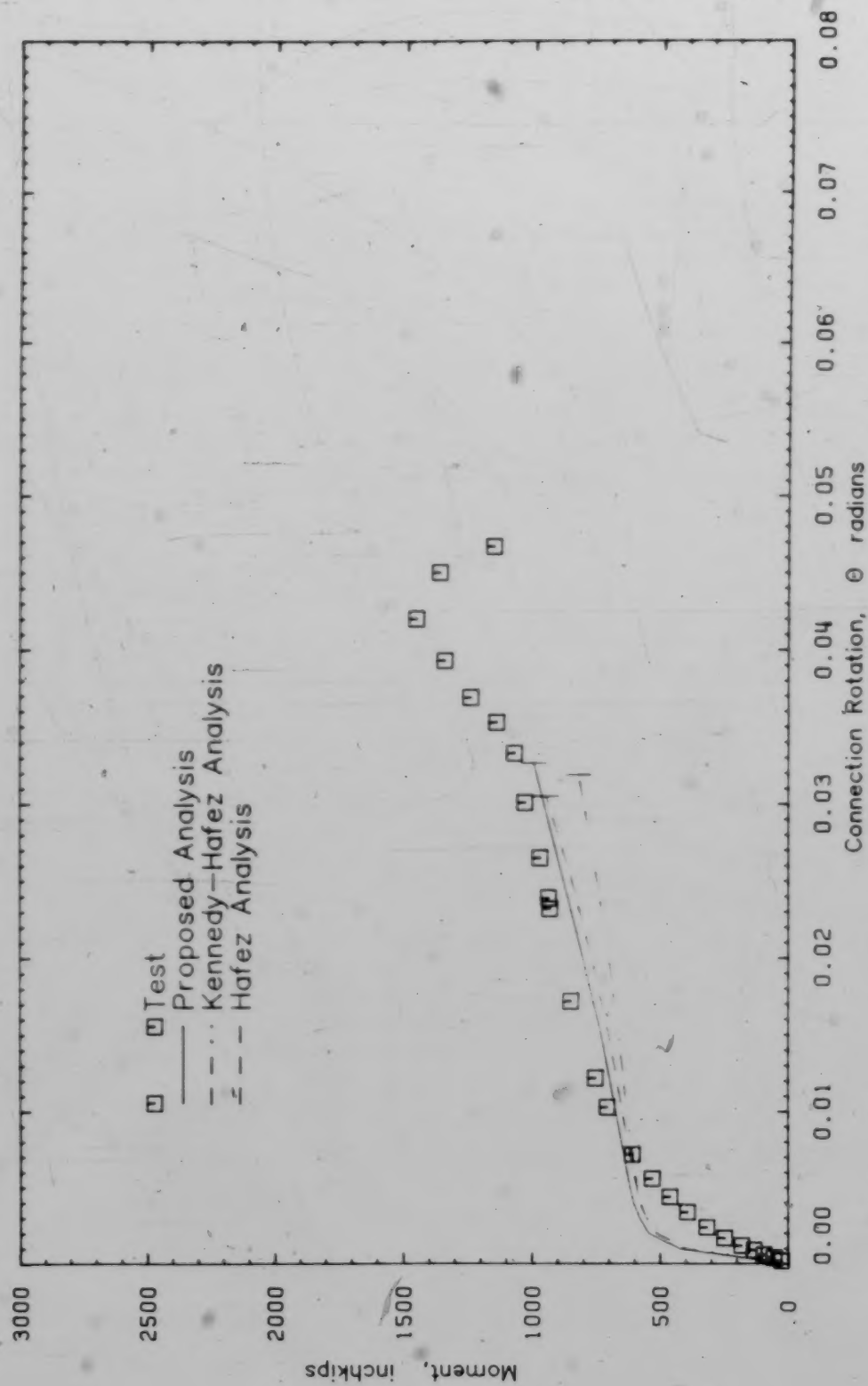


Figure 3.27 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 9

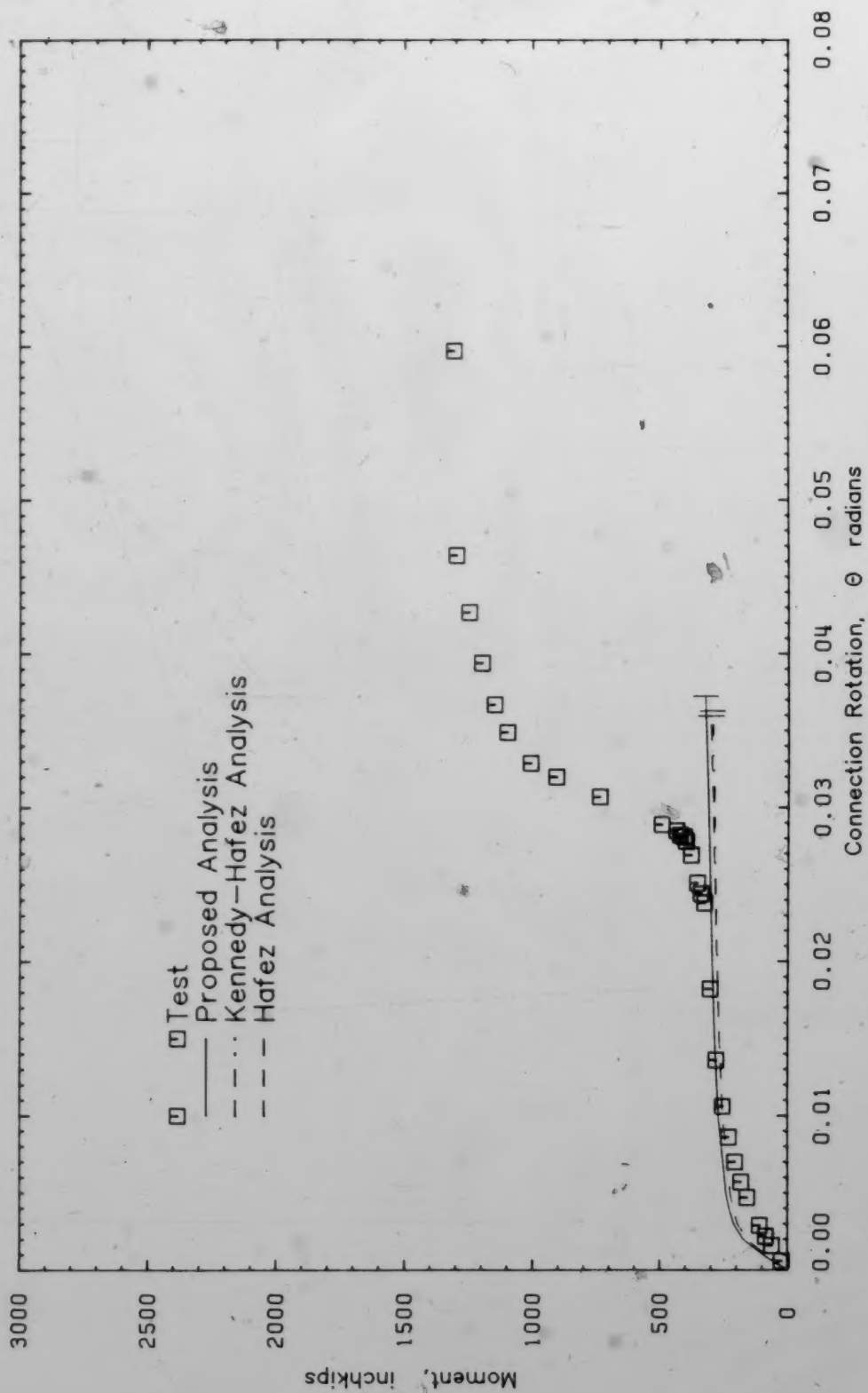


Figure 3.28 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 10

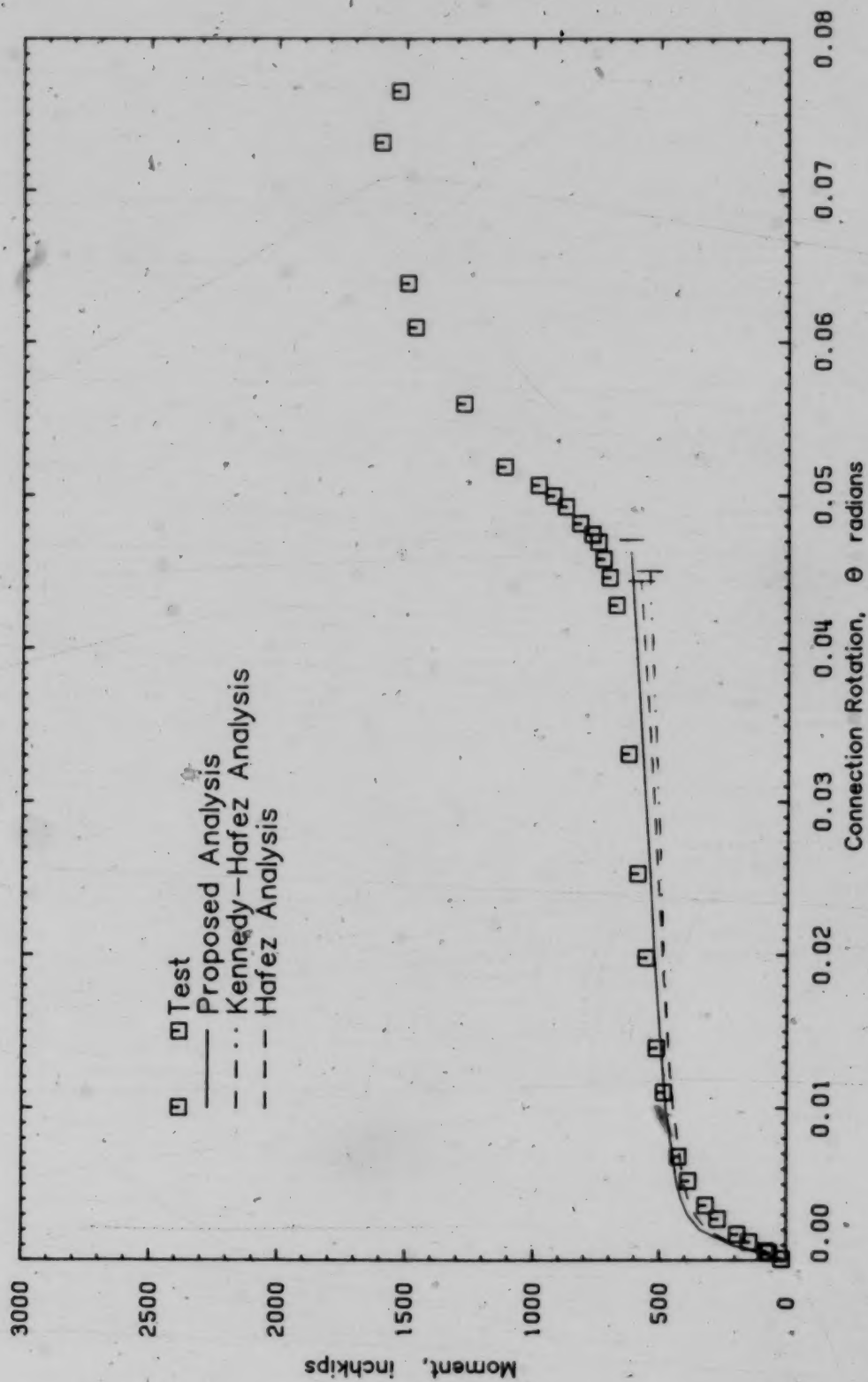


Figure 3.29 Comparison of Analytical Moment-Rotation Relationships
with Sommer (1969) End Plate Test # 11

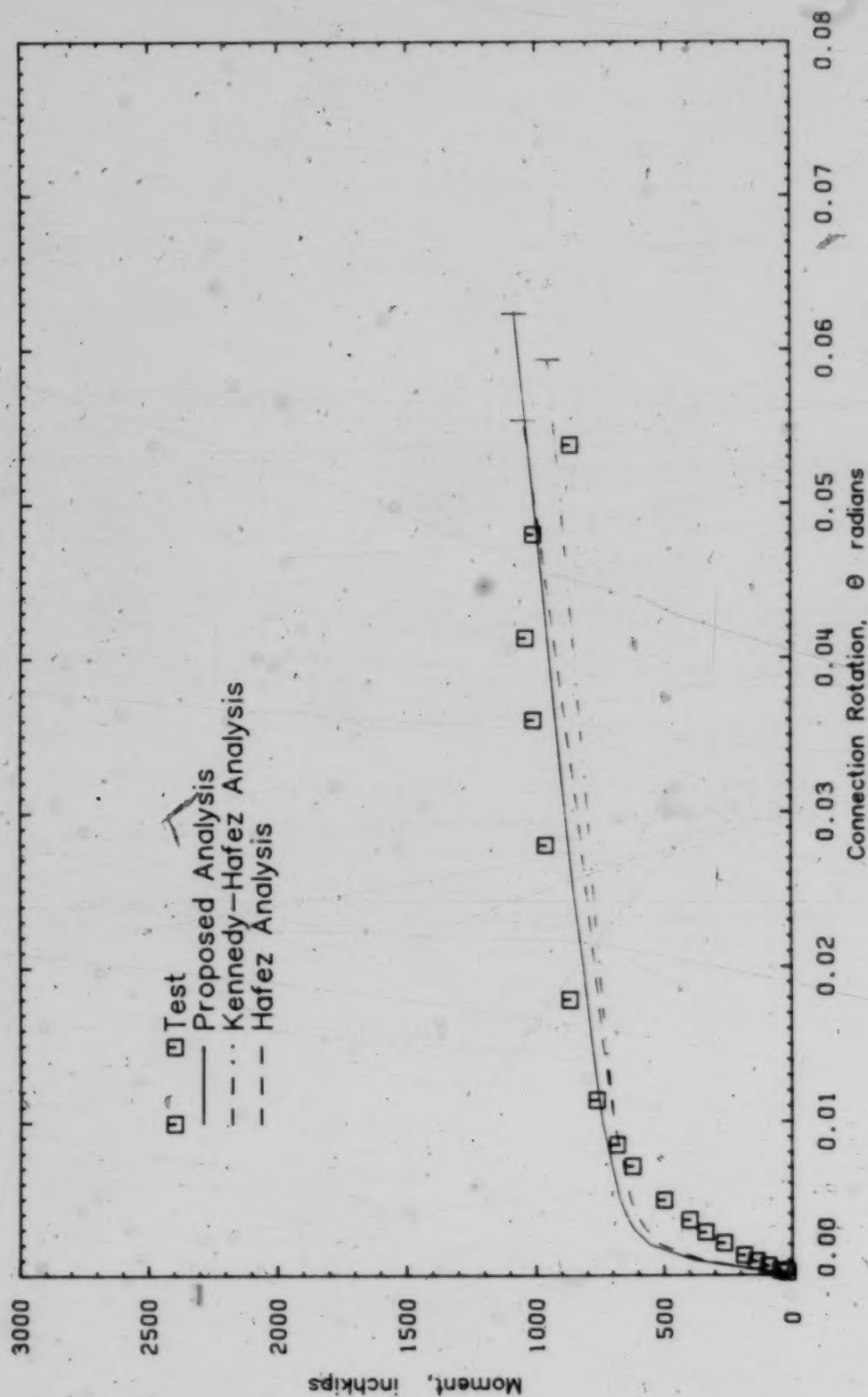


Figure 3.30 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 12

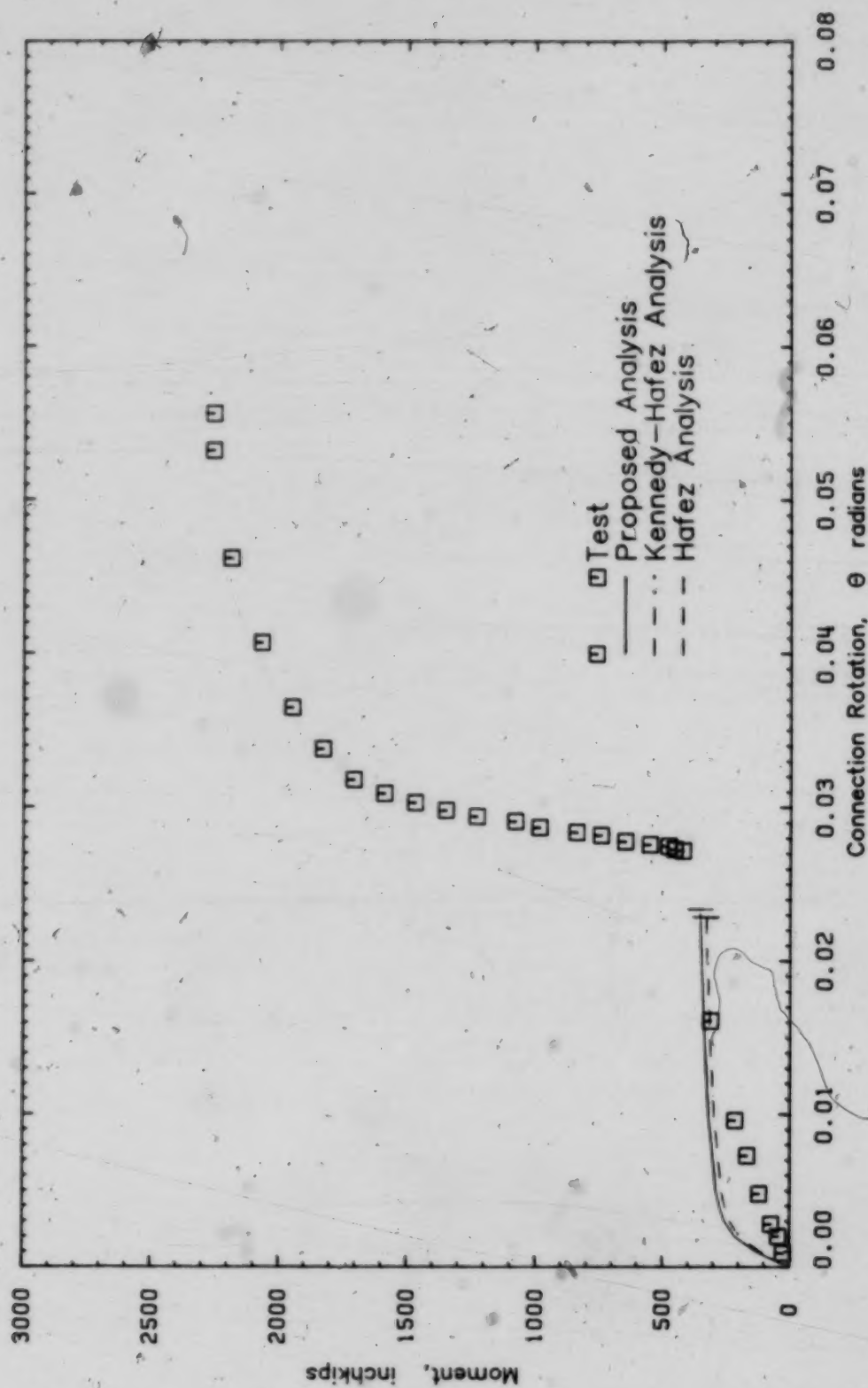


Figure 3.31 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 13

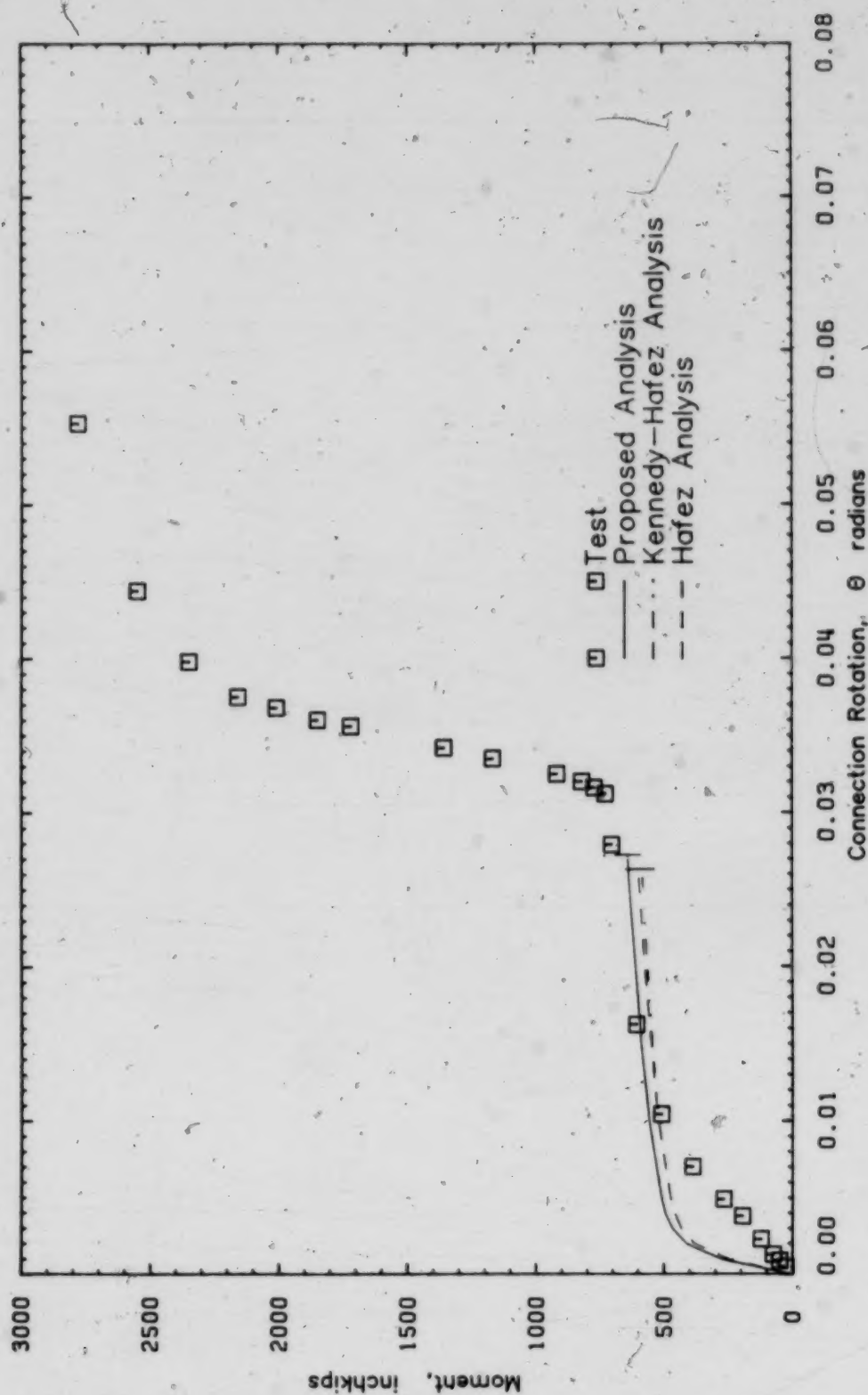


Figure 3.32 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 14

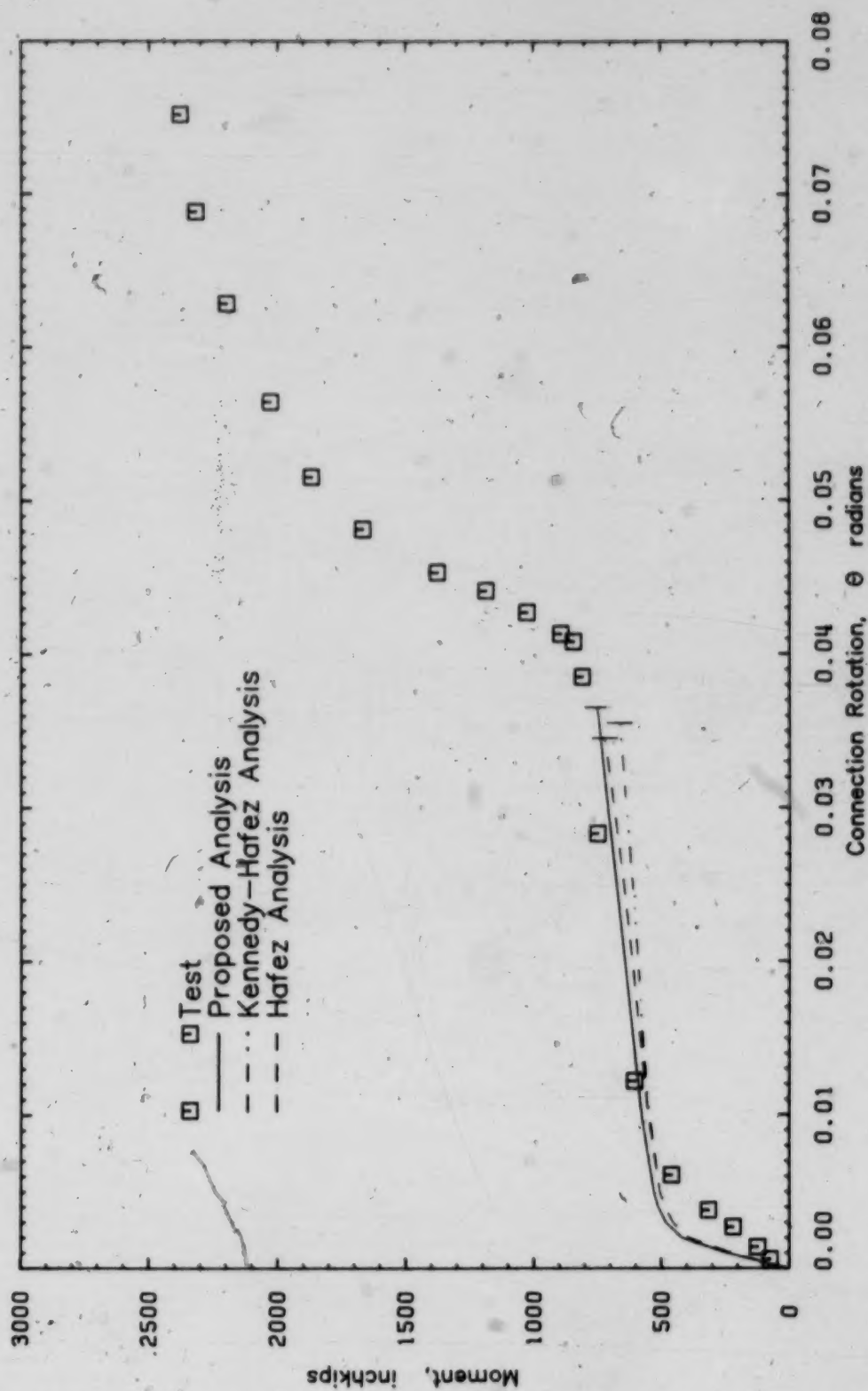


Figure 3.33 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 15

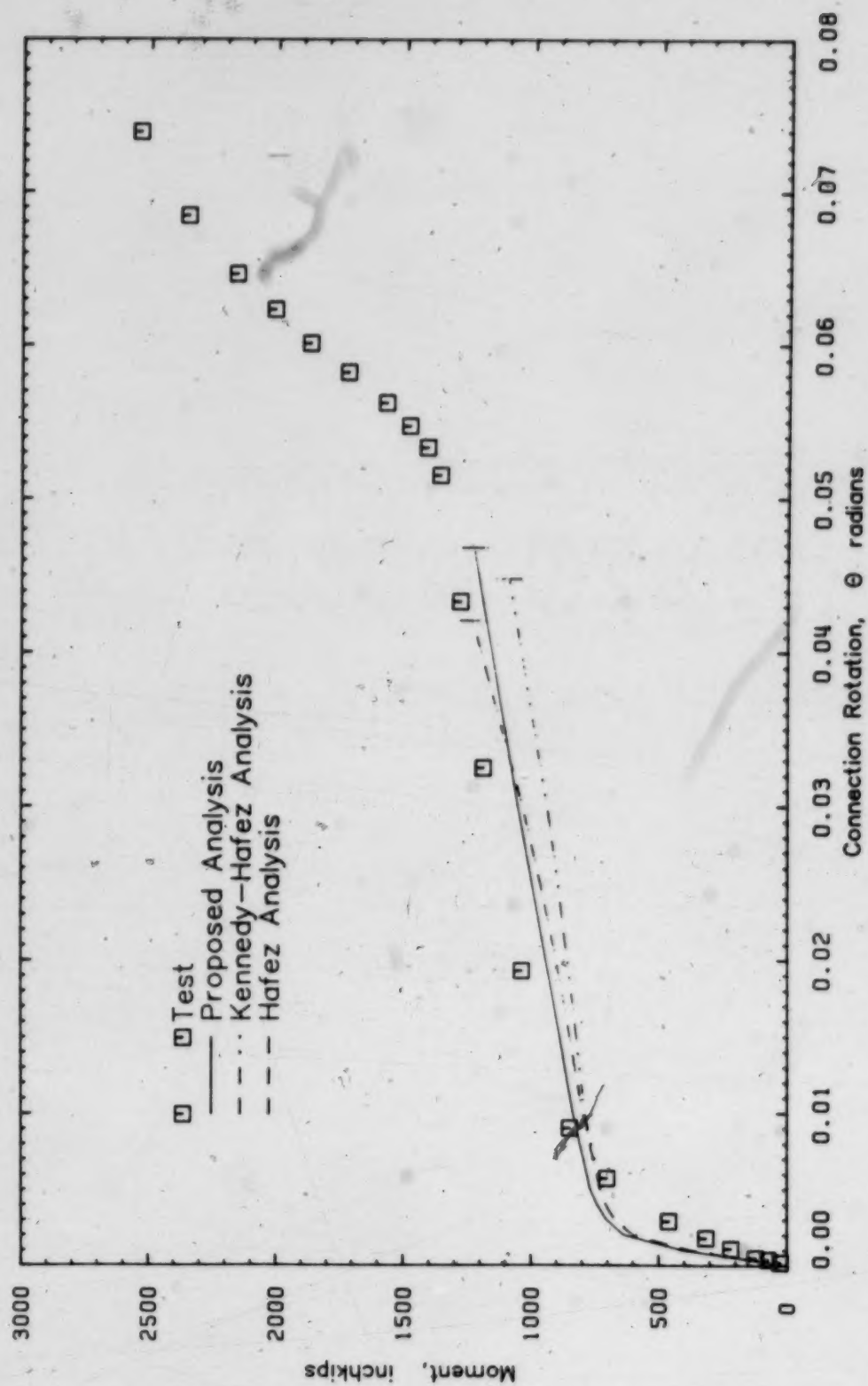


Figure 3.34 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 16

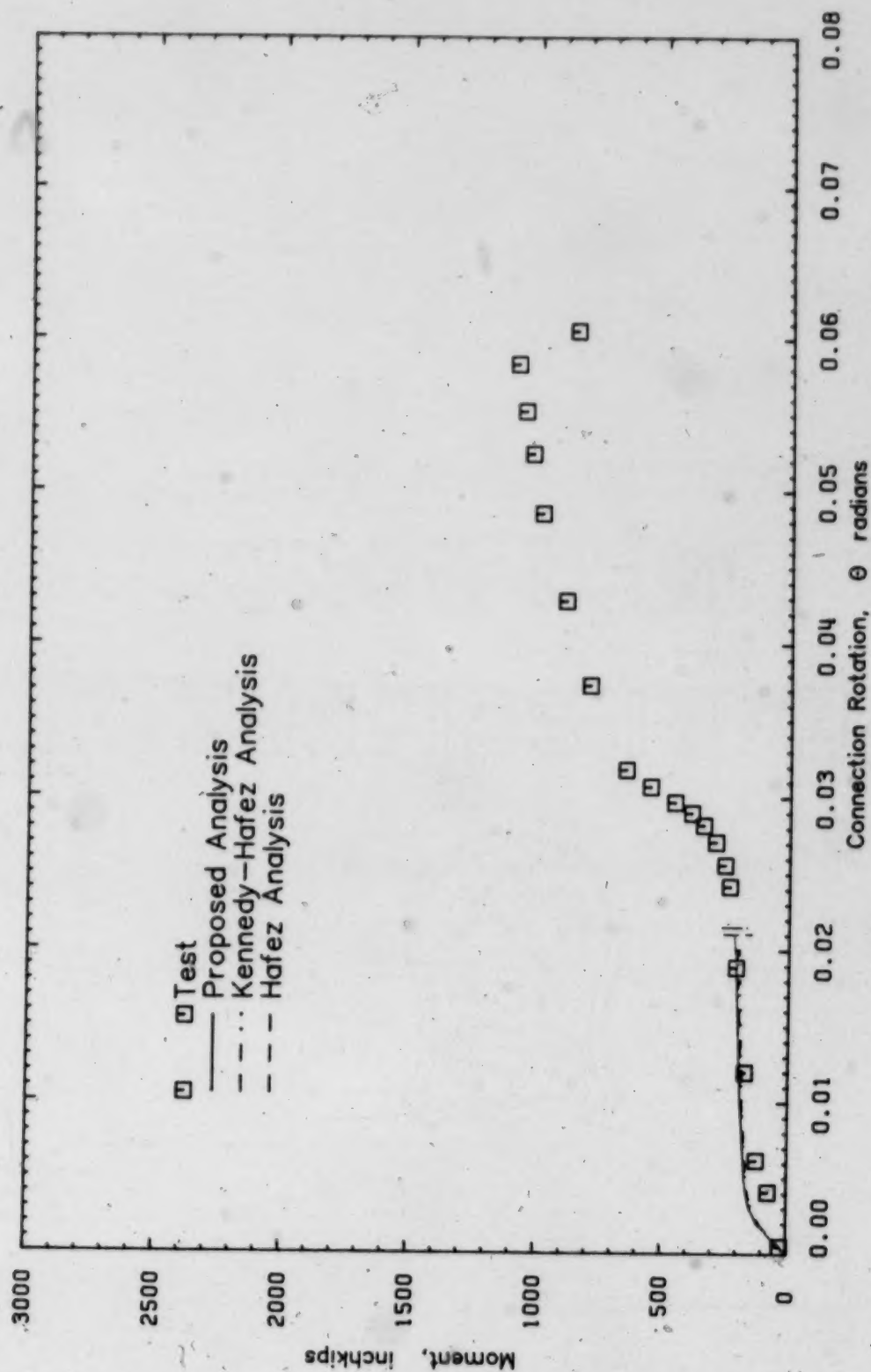


Figure 3.35 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 17

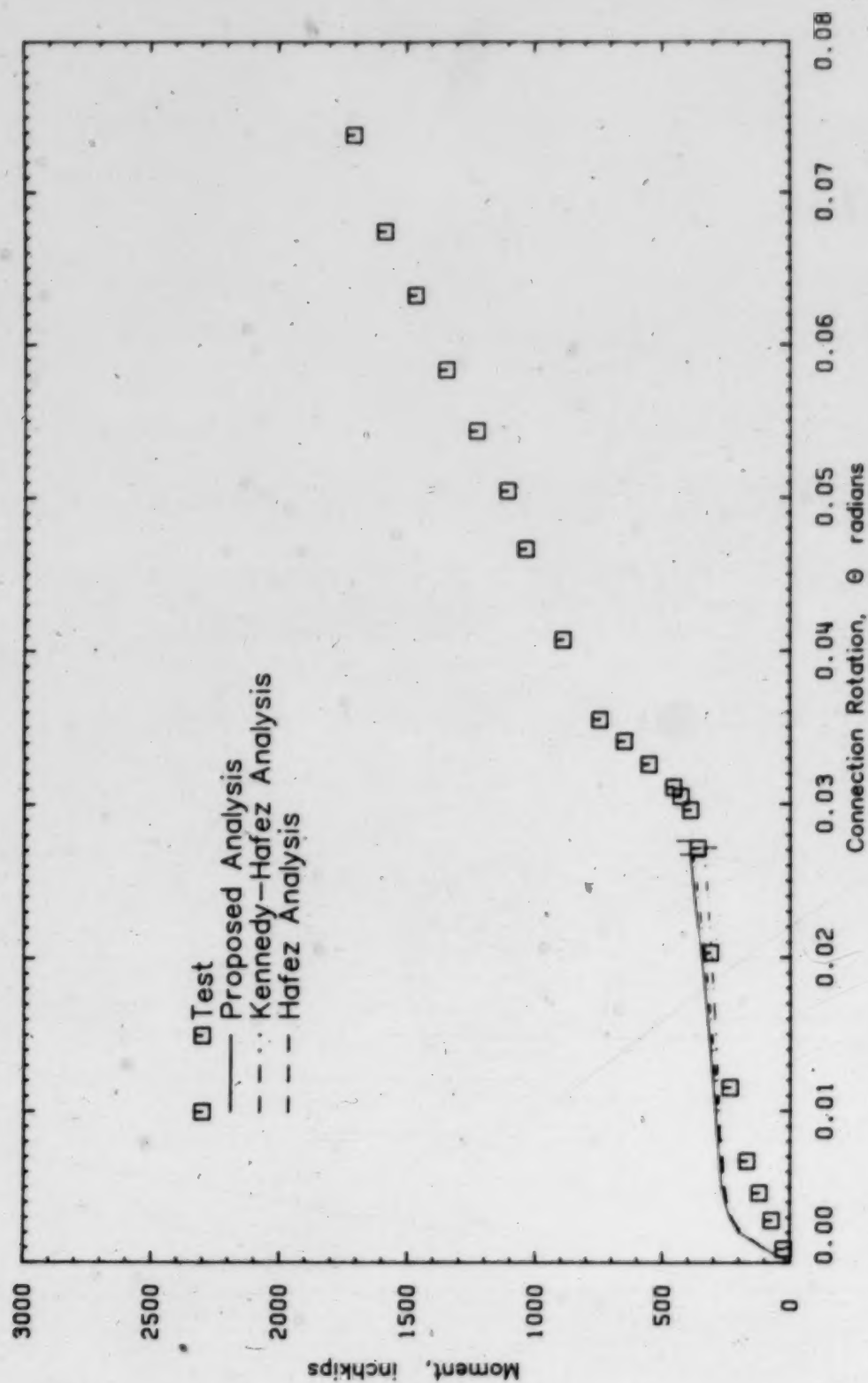


Figure 3.36 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 18

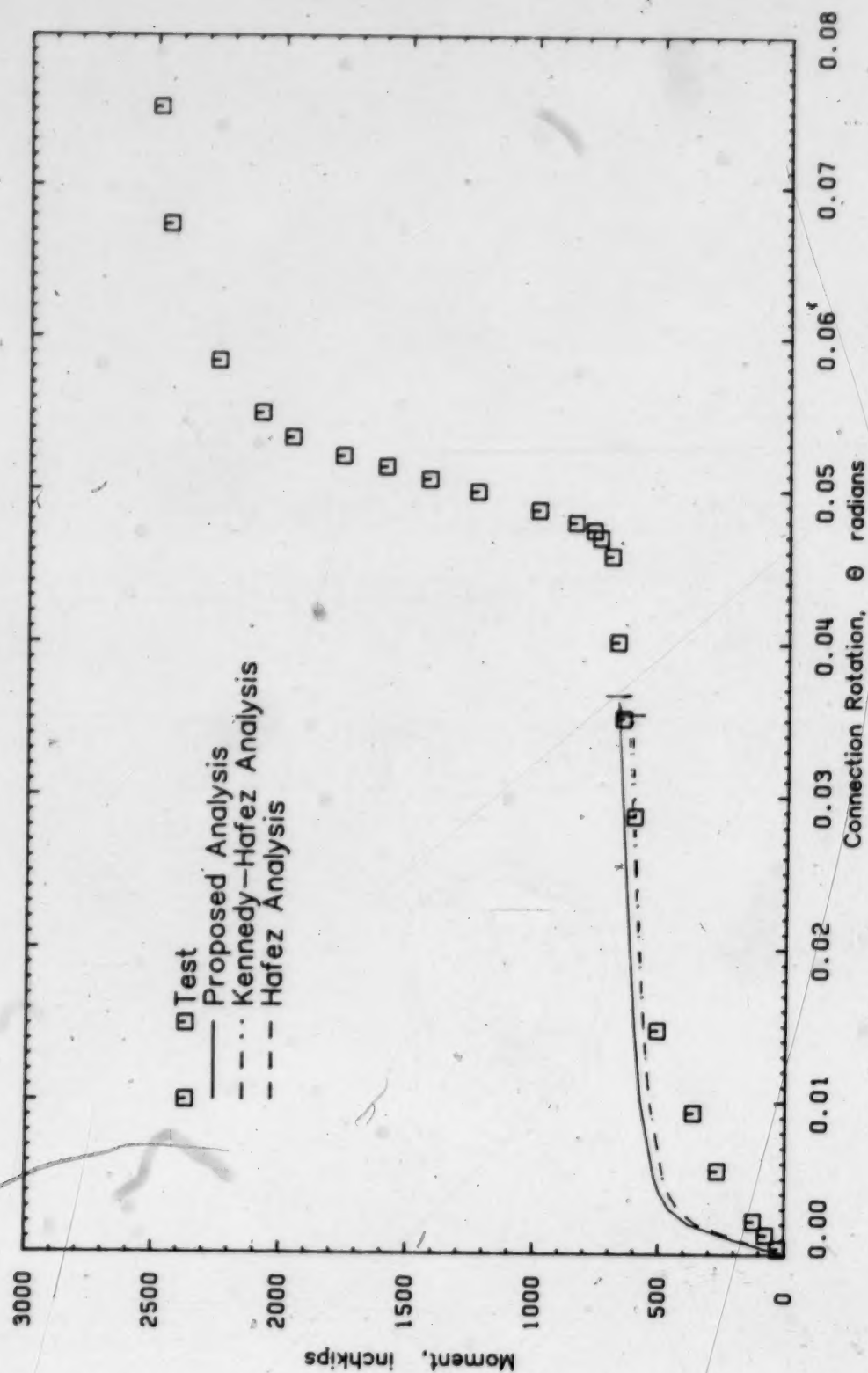


Figure 3.37 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 19

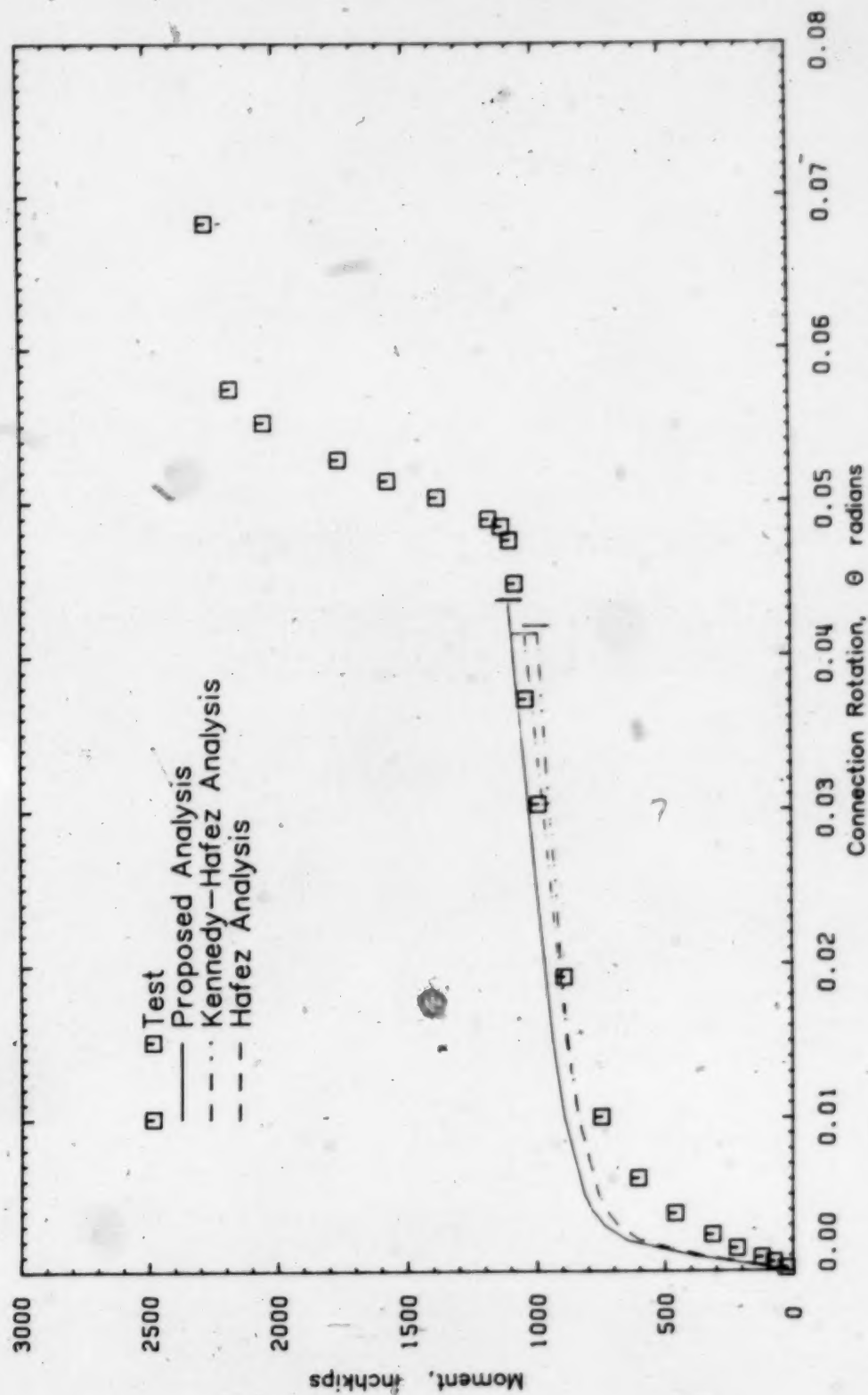


Figure 3.38 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 20

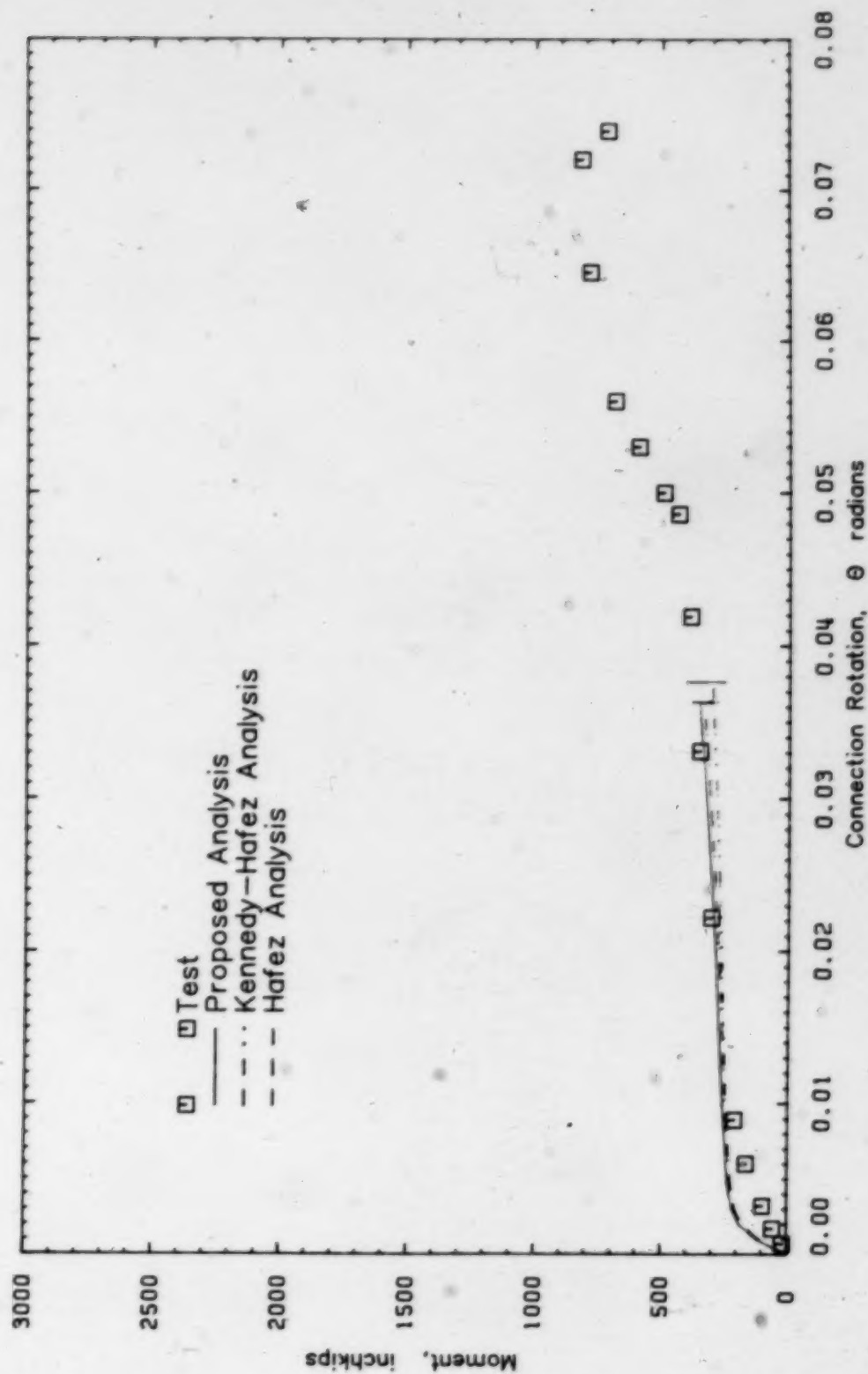


Figure 3.39 Comparison of Analytical Moment-Rotation Relationships with Sommer (1989) End Plate Test # 25

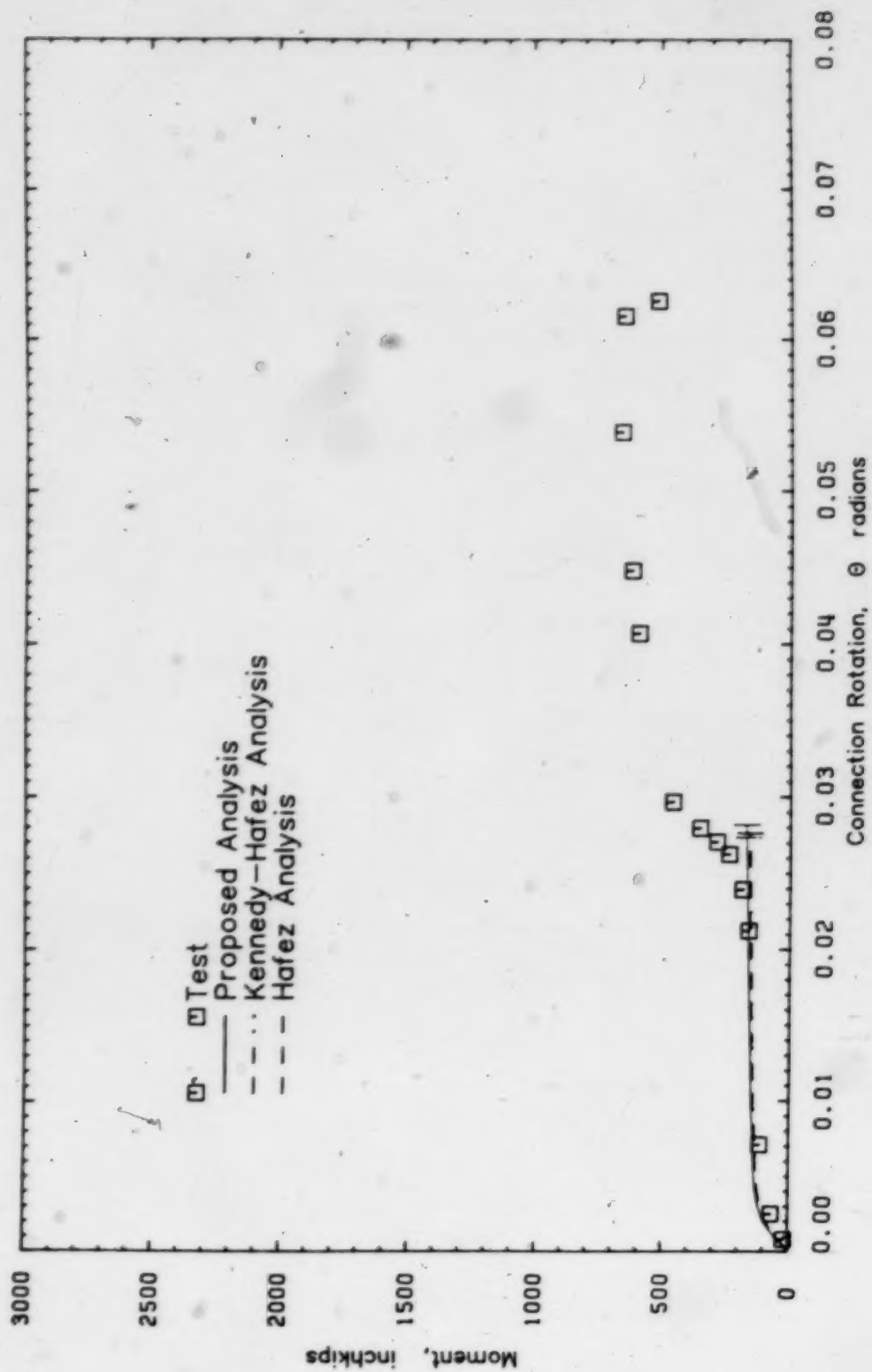


Figure 3.40 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 26

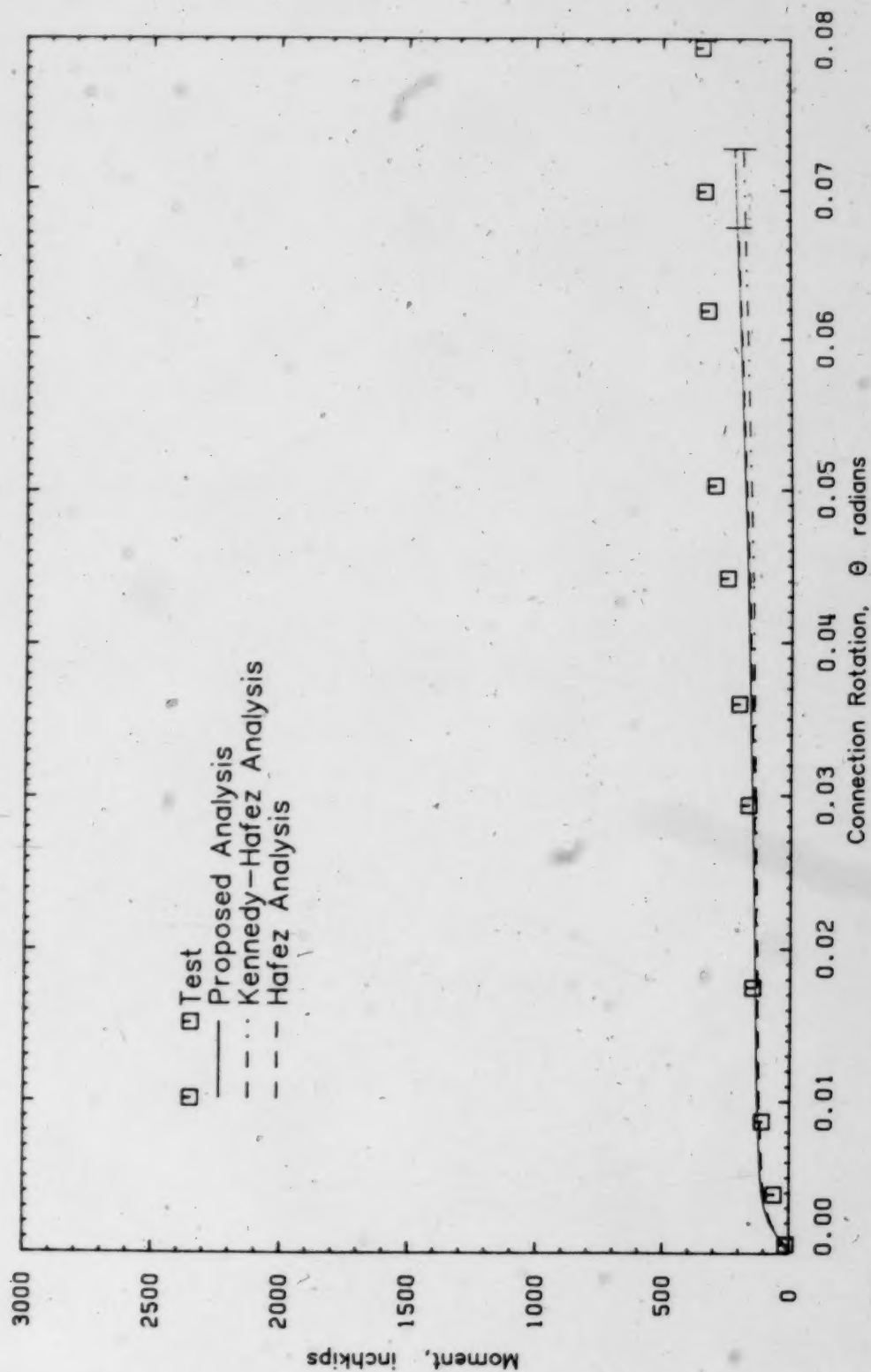


Figure 3.41 Comparison of Analytical Moment-Rotation Relationships with Sommer (1969) End Plate Test # 27

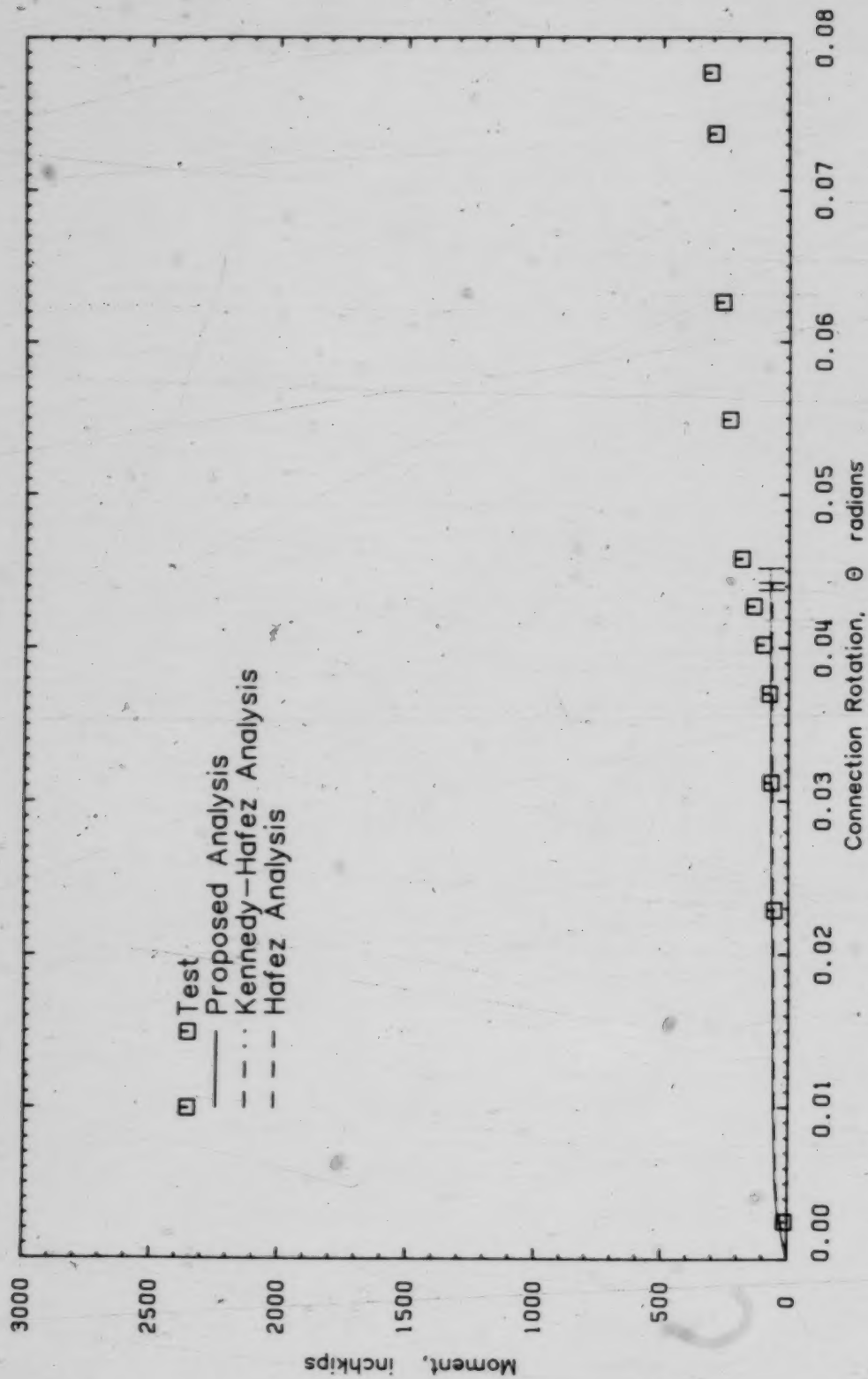


Figure 3.42 Comparison of Analytical Moment-Rotation Relationships
with Sommer (1969) End Plate Test # 28

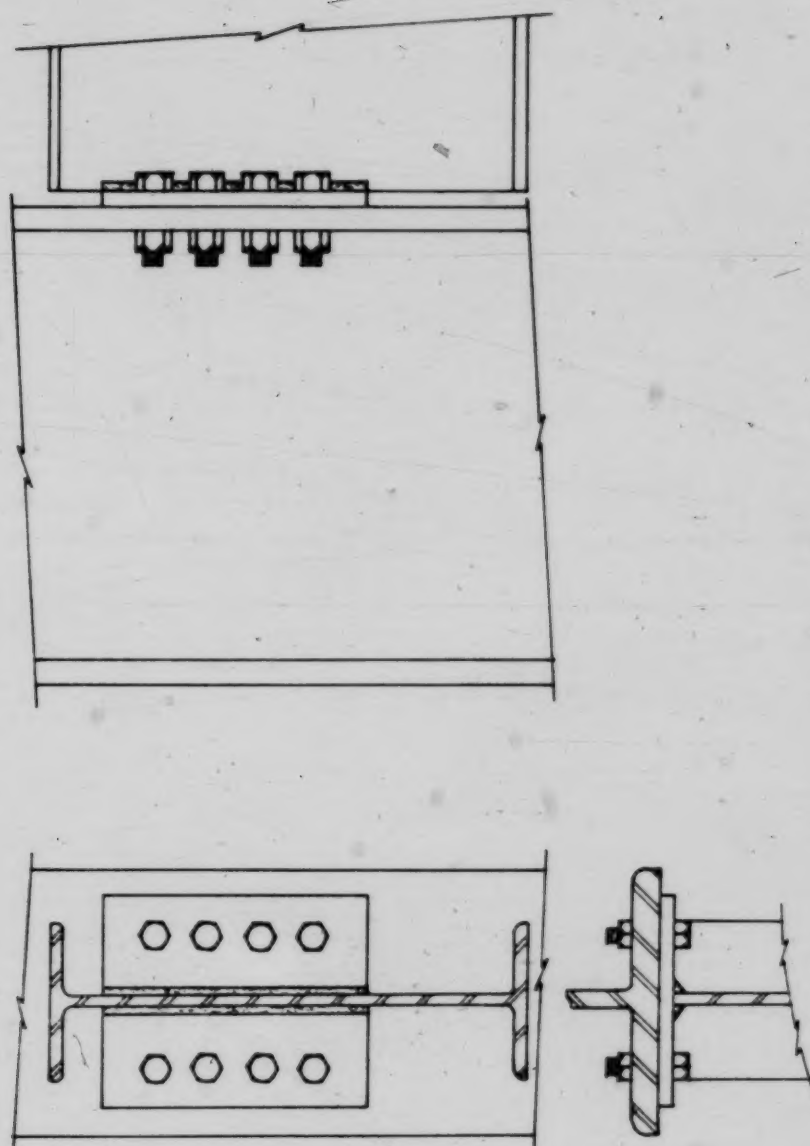


Figure 3.43 Flexible End Plate Connection to Column Flange Detail

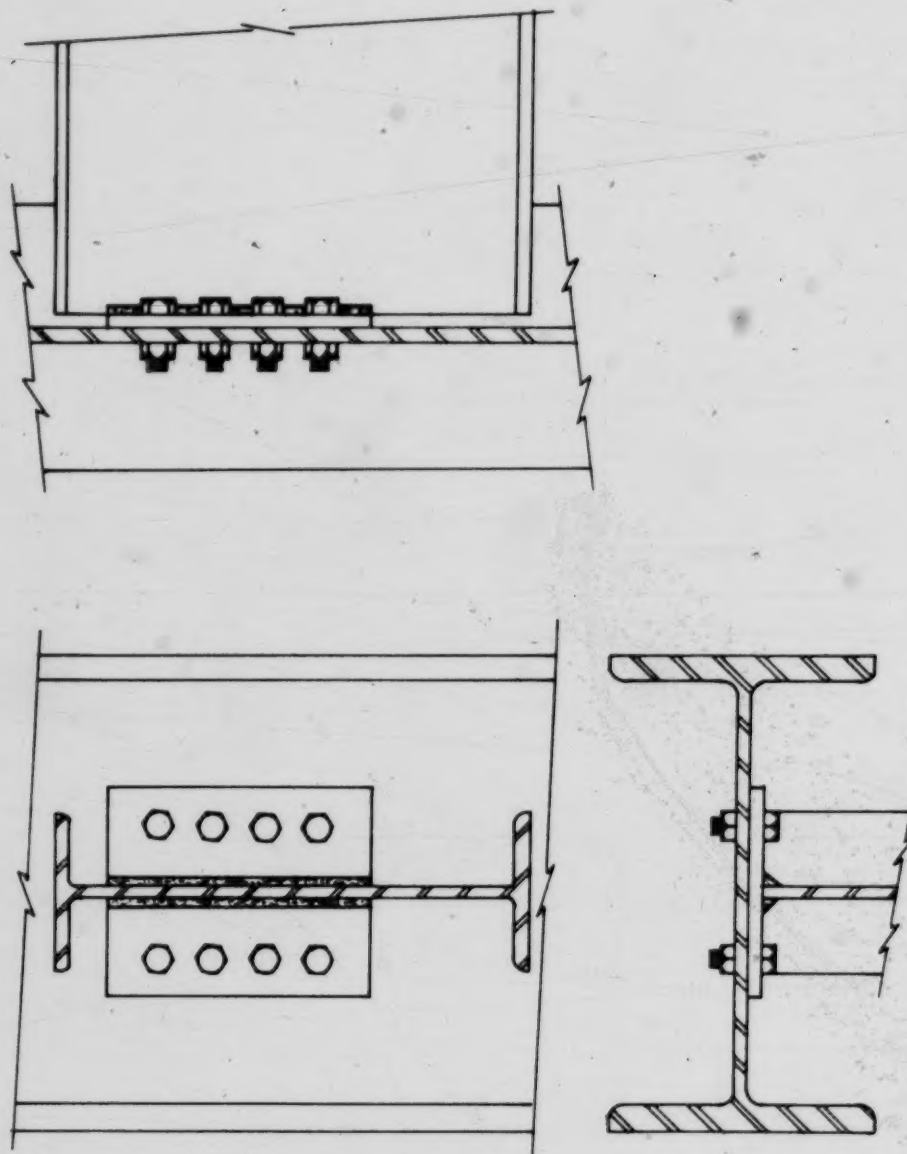


Figure 3.44 Single Flexible End Plate Connection to Column Web Detail

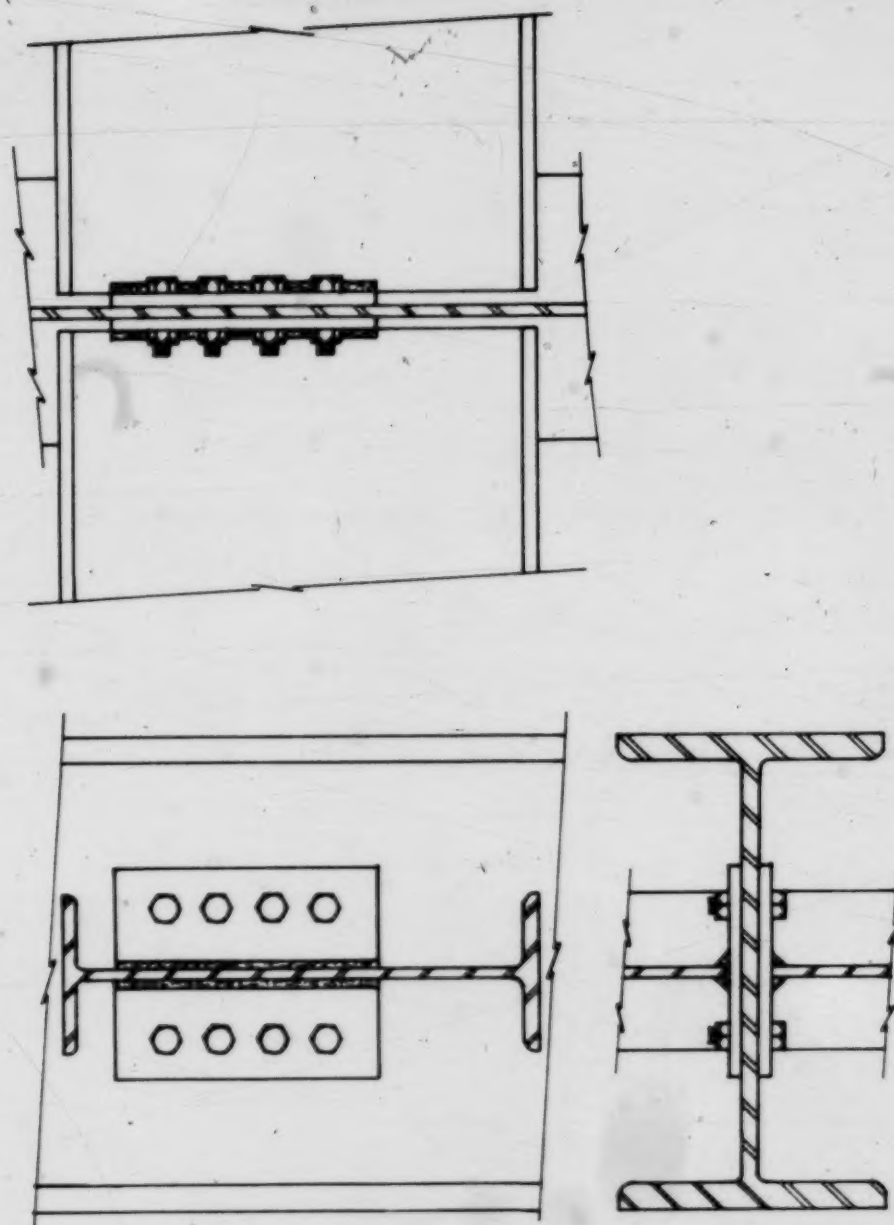


Figure 3.45 Double Flexible End Plate Connection to Column Web Detail

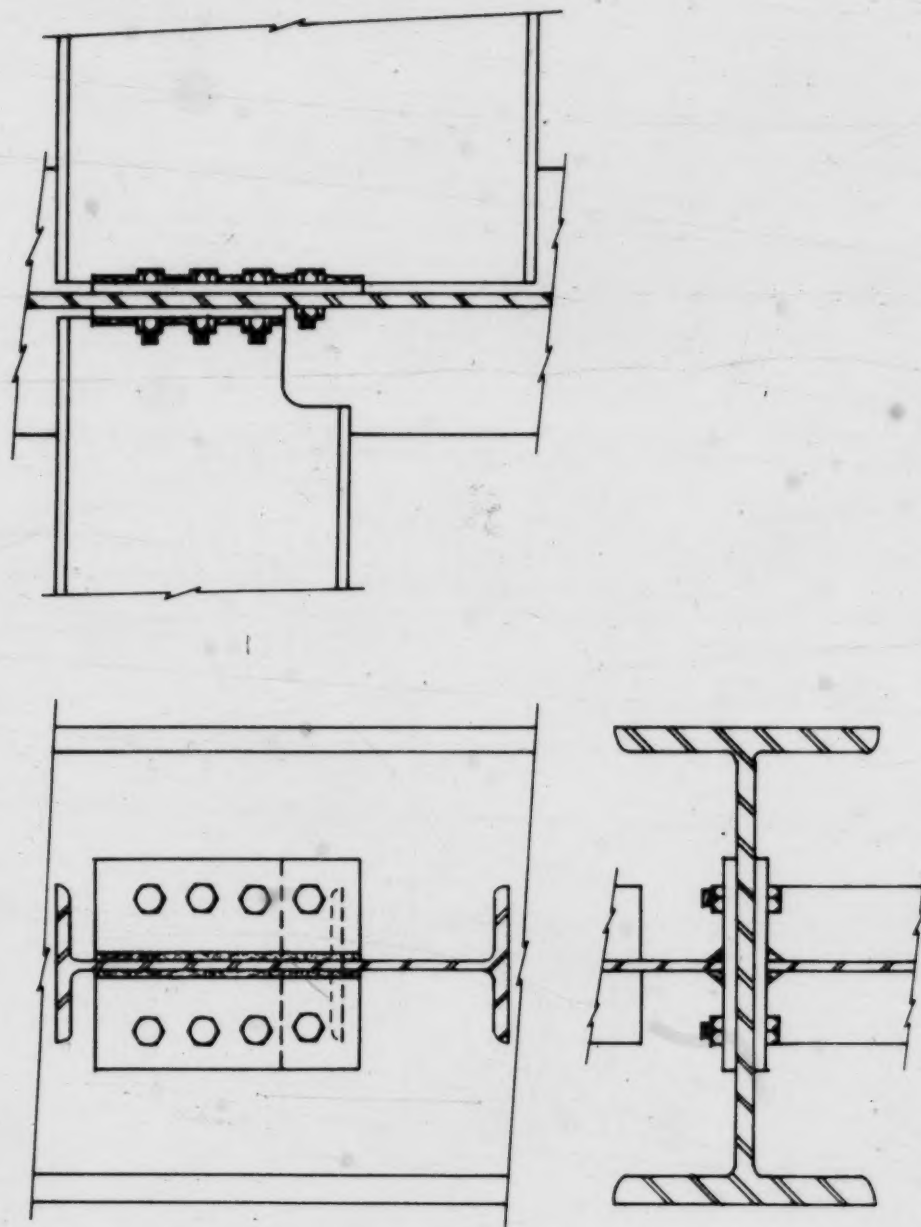


Figure 3.46 Double Flexible End Plate Connection to Column Web Detail

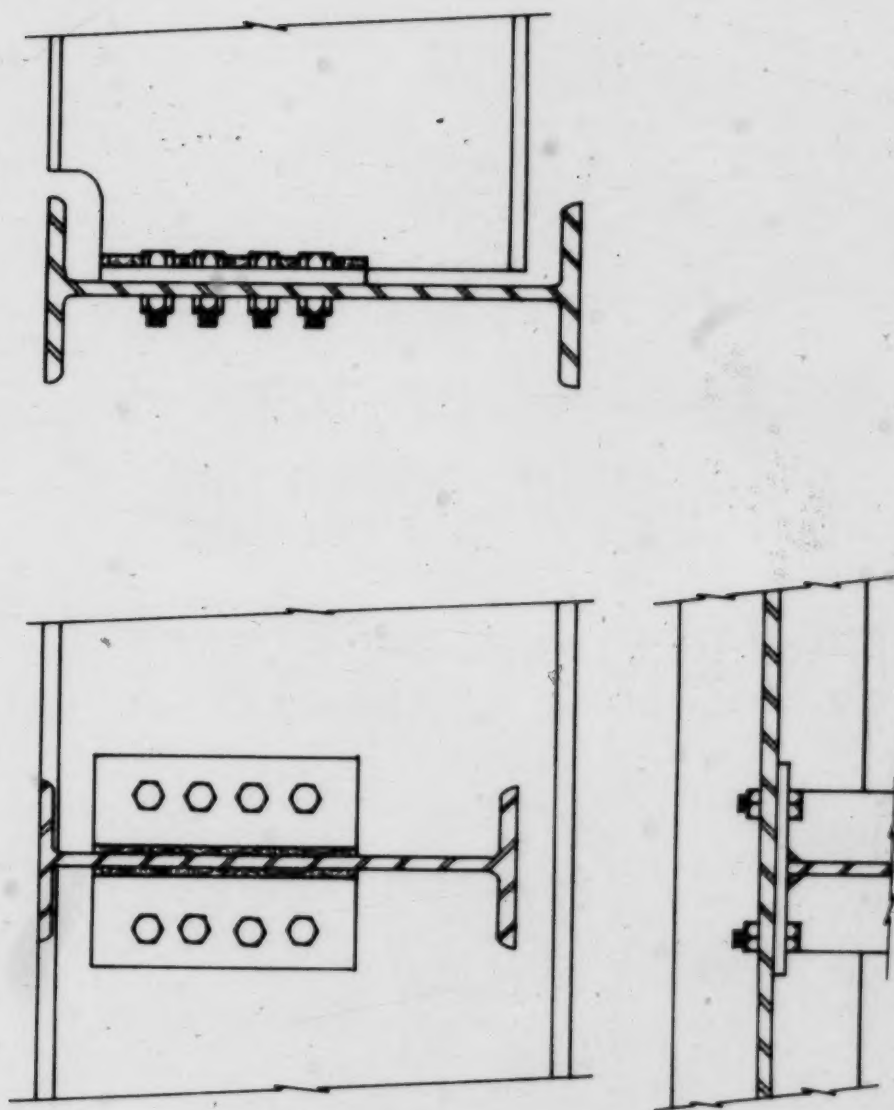


Figure 3.47 Single Flexible End Plate Connection to Beam Web Detail

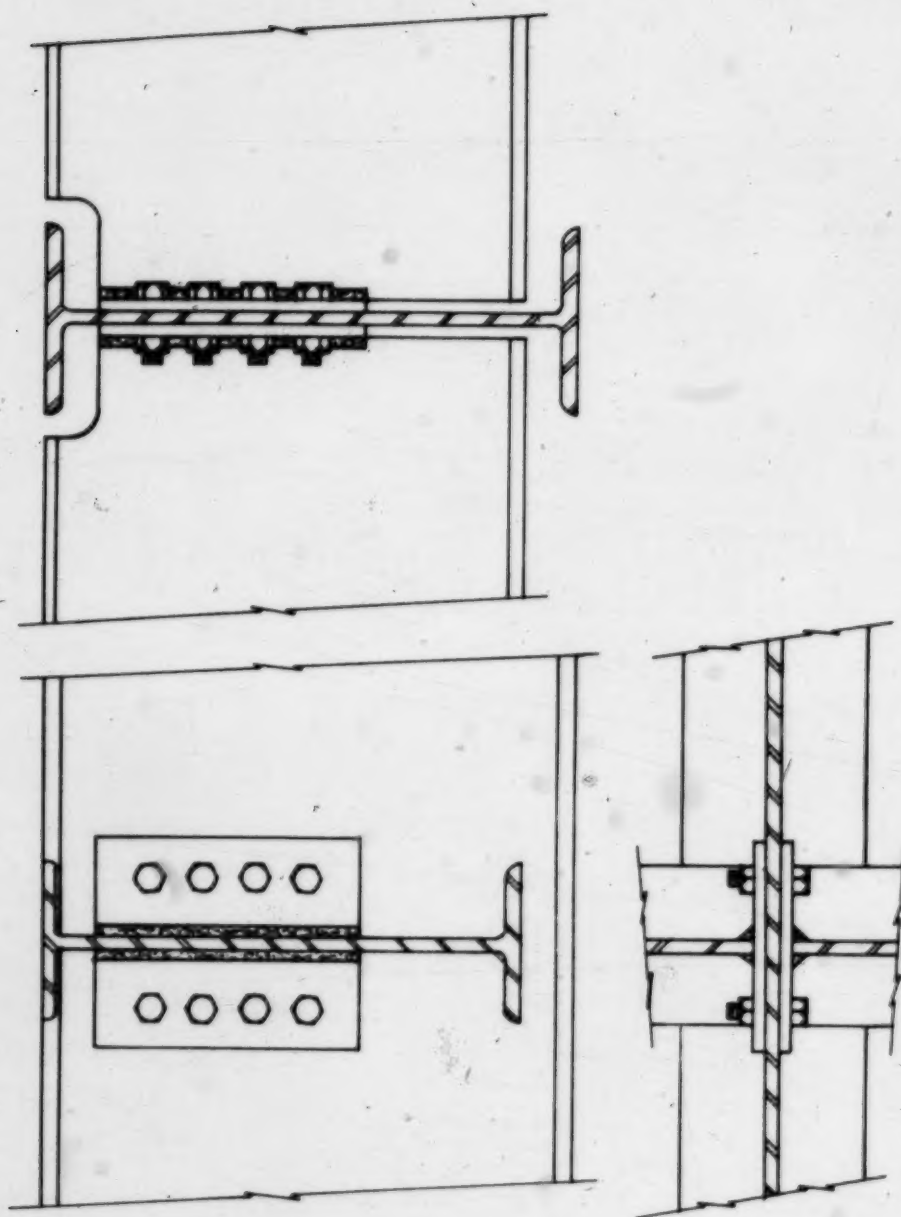


Figure 3.48 Double Flexible End Plate Connection to Beam Web Detail

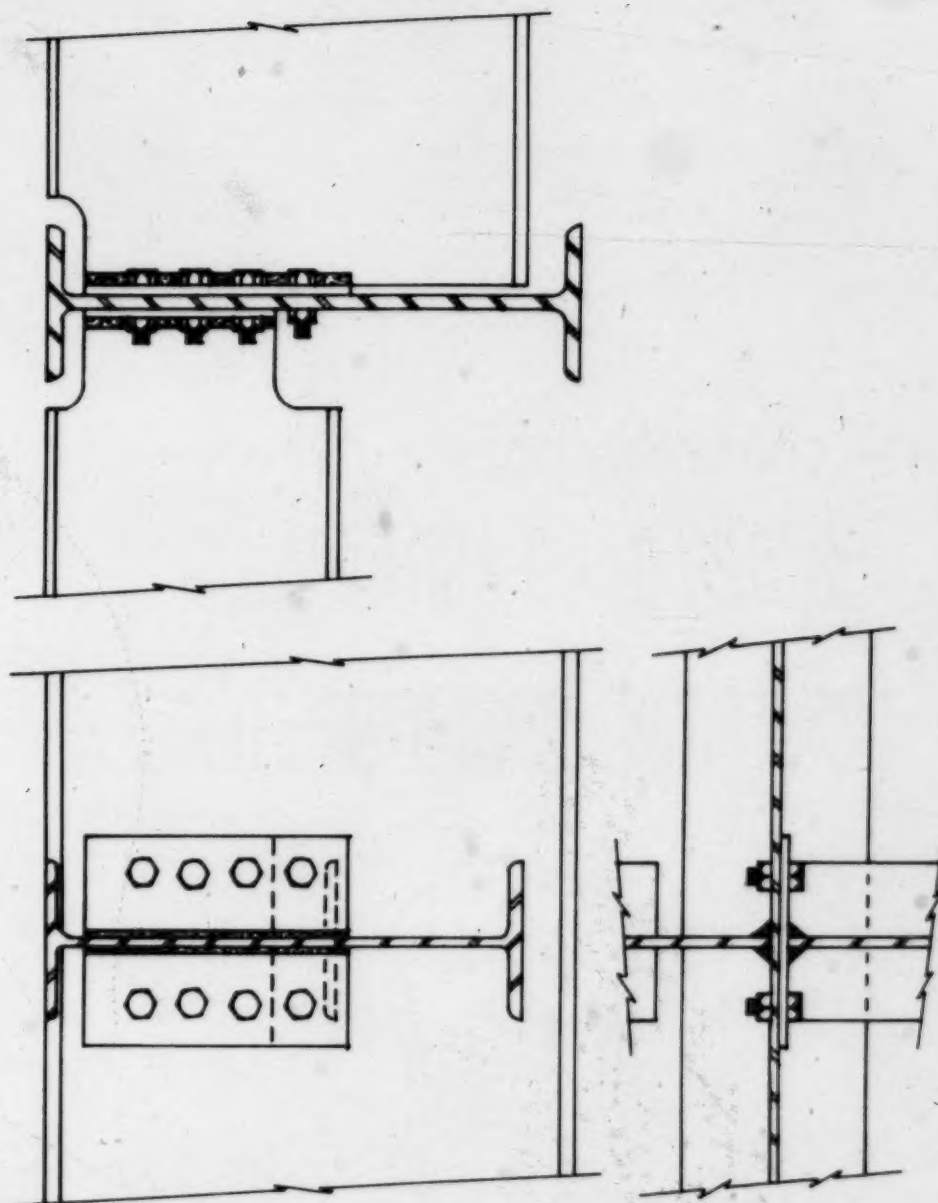


Figure 3.49 Double Flexible End Plate Connection to Beam Web Detail

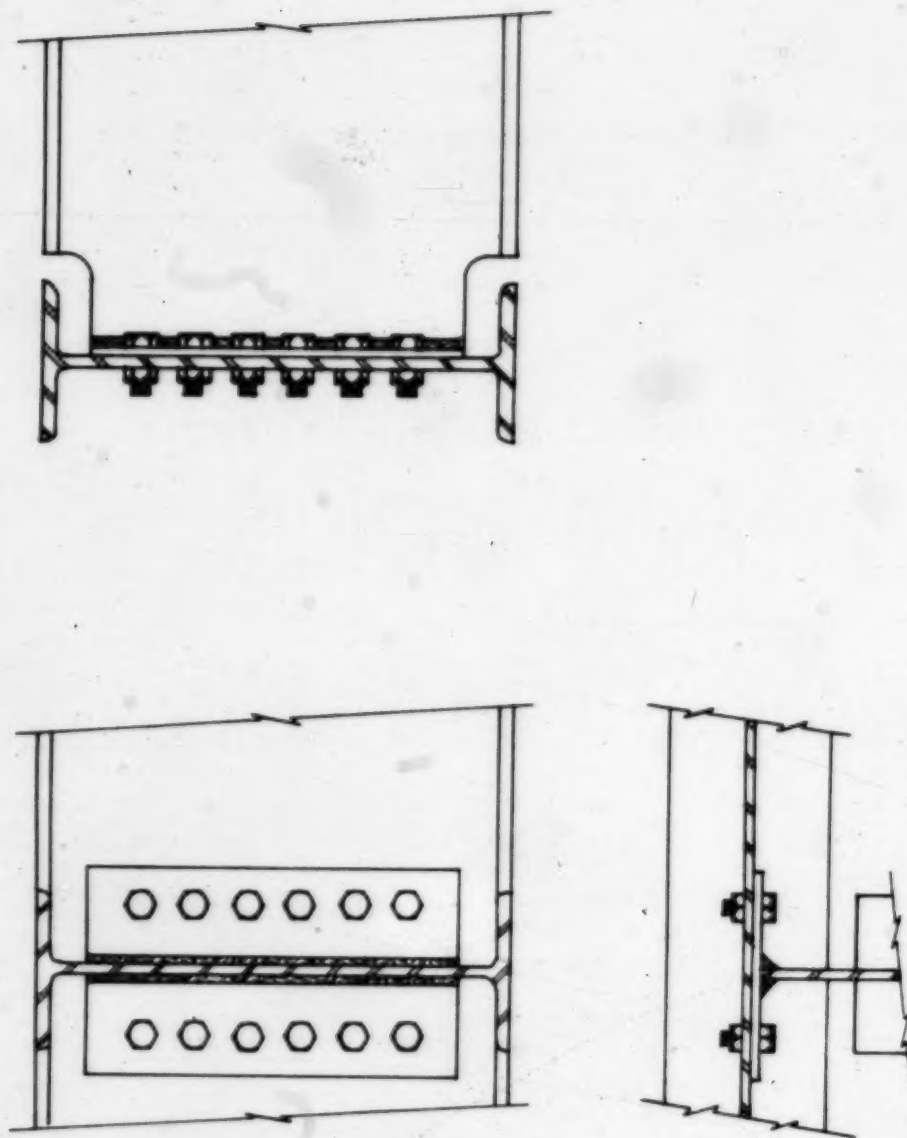


Figure 3.50 Single Flexible End Plate Connection to Beam Web Detail

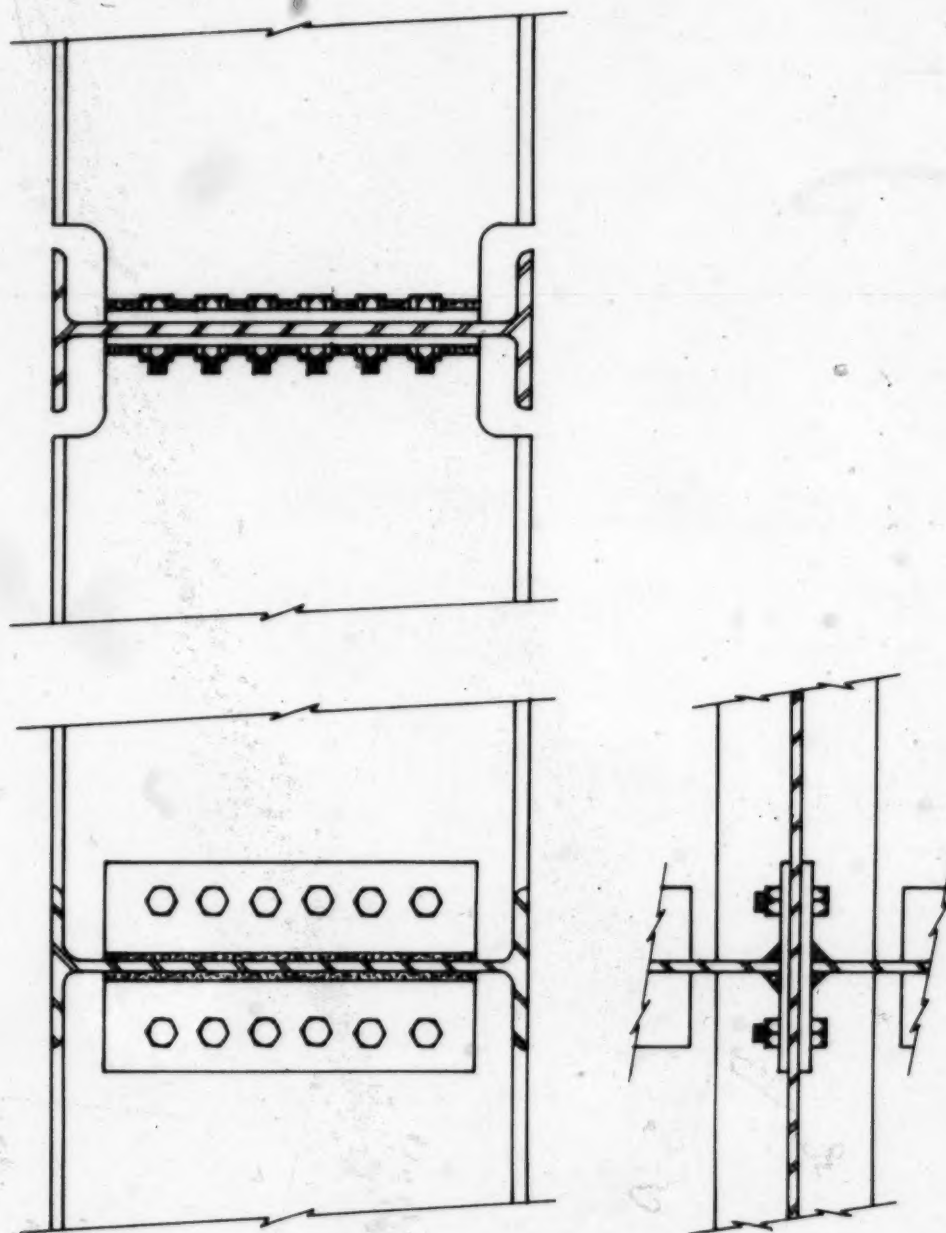


Figure 3.51 Double Flexible End Plate Connection to Beam Web Detail

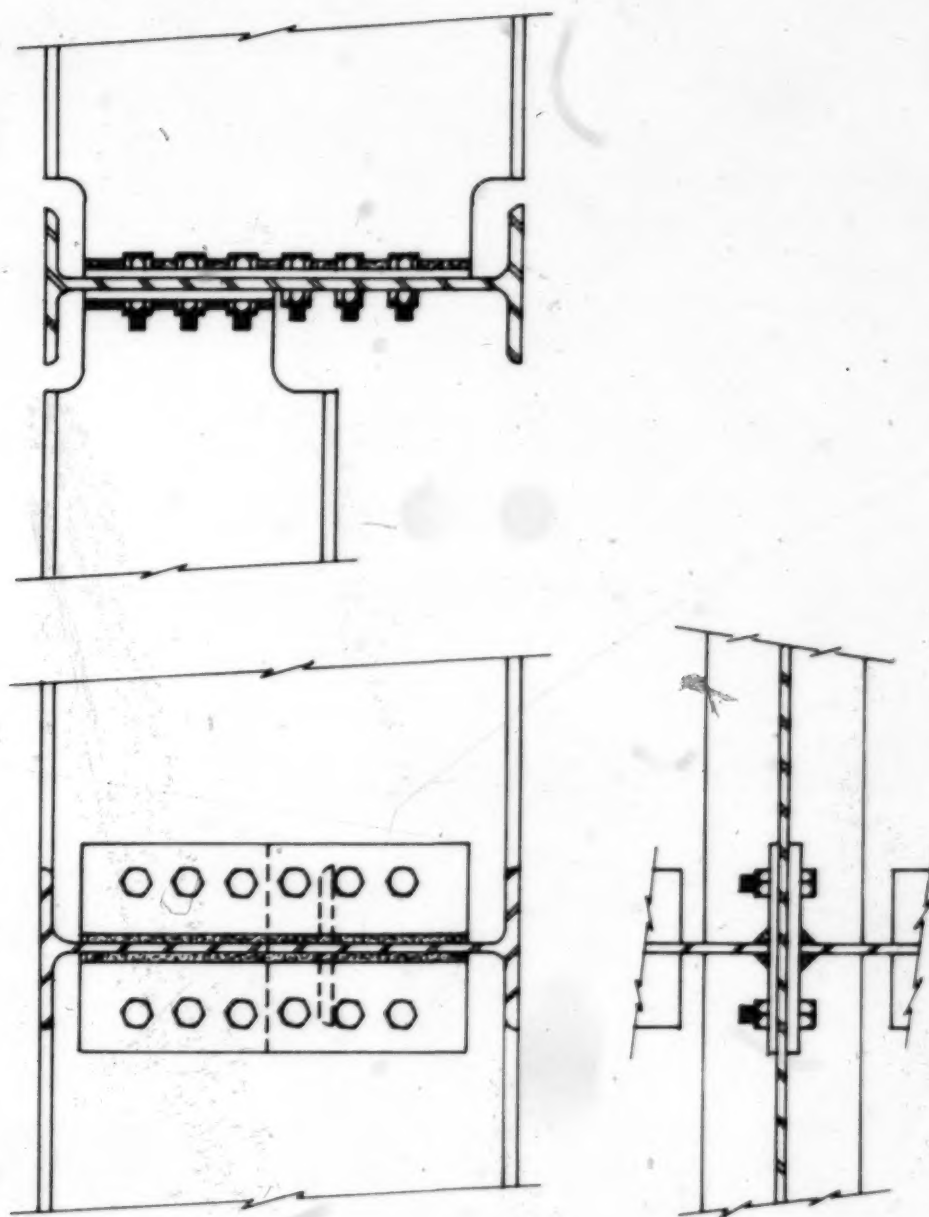


Figure 3.52 Double Flexible End Plate Connection to Beam Web Detail

4. Evaluation of the Unrestrained Rotation Limit of a Flexible End Plate Connection

4.1 Introduction

The flexible end plate connection is selected for its hinge-like characteristics, that is, the transmission of shear with the development of limited moments. However, the bottom flange can come to bear against the supporting member, as depicted in Figure 4.1, if the connection depth is selected only to provide sufficient resistance to primary shear forces, disregarding the rotation that the supported member end undergoes.

When bottom flange bearing occurs, the relatively large stiffness of the bottom flange of the supported member causes the neutral axis of the connection to move to a position near the bottom of the supported beam, about which subsequent connection rotation occurs. The connection, with increased effective depth, is much stiffer flexurally as observed by both Sommer (1969) and Hafez (1982).

The tensile deformation of the top part of the end plate also increases rapidly. If the magnitude of the tensile forces in the connection become too large, the primary shear capacity of either the welds, the plate, or the bolts may be diminished so that shear failure of the connection is imminent. Unrestrained connection rotation is essential if the flexible end plate is to perform as intended.

4.2 Analytical Method

Bottom flange bearing of the supported beam is imminent when the bottom surface of the flange has approached the support by a distance equal to the thickness of the end plate (Hafez 1982). From Figure 4.1, this condition occurs when the beam end rotation is:

$$[4.1] \quad \theta_b = \tan^{-1} \left(\frac{t_p}{l_b} \right)$$

For the angles of end rotation encountered in standard steel framing members, it is generally sufficient to calculate θ_b as:

$$[4.2] \quad \theta_b = \frac{t_p}{l_b}$$

Assuming a connection rotation θ_b in Equation [4.2], the neutral axis position is calculated. Then, applying the end plate load-deformation relationships over the entire depth of the connection, it is possible to check whether or not statics is satisfied. This procedure is repeated until the horizontal forces sum to zero.

4.3 Comparison of Analytical Method With Test Results

Even though the analytical methods presented here predict the shape of the moment-rotation curves well, the angle of rotation of the connection at bottom flange bearing, θ_b , is not predicted as well, as can be deduced

from Figures 3.15 to 3.42.

In two of the eight tests performed by Hafez (Figures 3.15 to 3.22) bottom flange bearing occurred at rotations less than those predicted by analysis, and in five of the tests the analysis underestimates the limiting rotation. Test # 8 cannot be considered valid since premature weld fracture rendered its behaviour to be non-typical.

In Sommer's test series, four of twenty tests experienced bottom flange bearing prior to the point predicted by analysis, and in fifteen tests the reverse was true. Test # 12 failed prematurely in the heat-affected zone and should not be included in analysis.

From Table 3.1, the mean test-to-predicted ratio of the rotation at column bearing by the Kennedy-Hafez analysis is 1.11 with a coefficient of variation of 0.25, while the proposed method gives a ratio of 1.08 and a coefficient of variation of 0.25. The dispersion as reflected by the coefficient of variation is considerably greater for the rotations than for the moments.

To account for the greater dispersion and to preclude, insofar as possible, premature bearing of the bottom flange of the supported member on the support, Hafez (1982) suggests using only $2/3$ of the predicted rotation when the method is used in design. Only one test, (Sommer (1969) Test # 27), falls below this value at a test-to-predicted value of 0.50. However, because of the flexible nature of this particular connection, the use of the $2/3$ rule would not

have resulted in too excessive an end moment, as can be deduced from Figure 3.41.

Bottom flange bearing is chiefly a problem when the support details are similar to those used in the tests of Sommer (1969) and Hafez (1982), as depicted in Figure 3.43. Bolt deformations, or other unaccounted for tensile deformations, would tend to lower the position of the neutral axis and, therefore, from Equation [4.2], increase the rotation at which bottom flange bearing would occur. If the supporting member deforms locally, near the bottom of the end plate, as shown in Figure 4.2, bearing would occur sooner than predicted.

In connection details which have the bottom flange of the supported member coped, as depicted in Figures 3.46, 3.49, 3.50, 3.51, and 3.52, connection rotation will not be restrained by bottom flange bearing for usual proportions and this condition need not be checked for such details.

Although generally not considered in design, the deflection of a beam at ultimate load levels causes a shortening of the beam to occur, as shown in Figure 4.3. If the supporting members are restrained from moving, the compressed region of the end plate connection would be reduced. This would tend to reduce the possibility of bottom flange bearing.

The presence of shear stresses in steel members loaded into the plastic range, is known to 'soften' the normal or flexural stress-strain response of such members (ASCE 1971).

It is likely that some degree of softening does occur in the web of the supported beam and in the end plate, because significant plastic behaviour is observed to occur in these regions (Sommer 1969, Hafez 1982). Since no account is made of this reduced stiffness, the analytical method would not be expected to predict θ_b accurately in all cases. Depending upon the distribution of shear stresses in the connection, the softening effect of the shear stresses may result in either or both a compression region or a tension region that does not follow exactly the load-deformation behaviour of the end plate details without such shear stresses.

It is apparent that quantifying the value of θ_b can be complicated by several factors associated with a connection detail. To account for these factors, which are not considered directly in the analysis, the proposal of Hafez (1982) to use 2/3 of the value of θ_b predicted by the analysis, is recommended for design use.

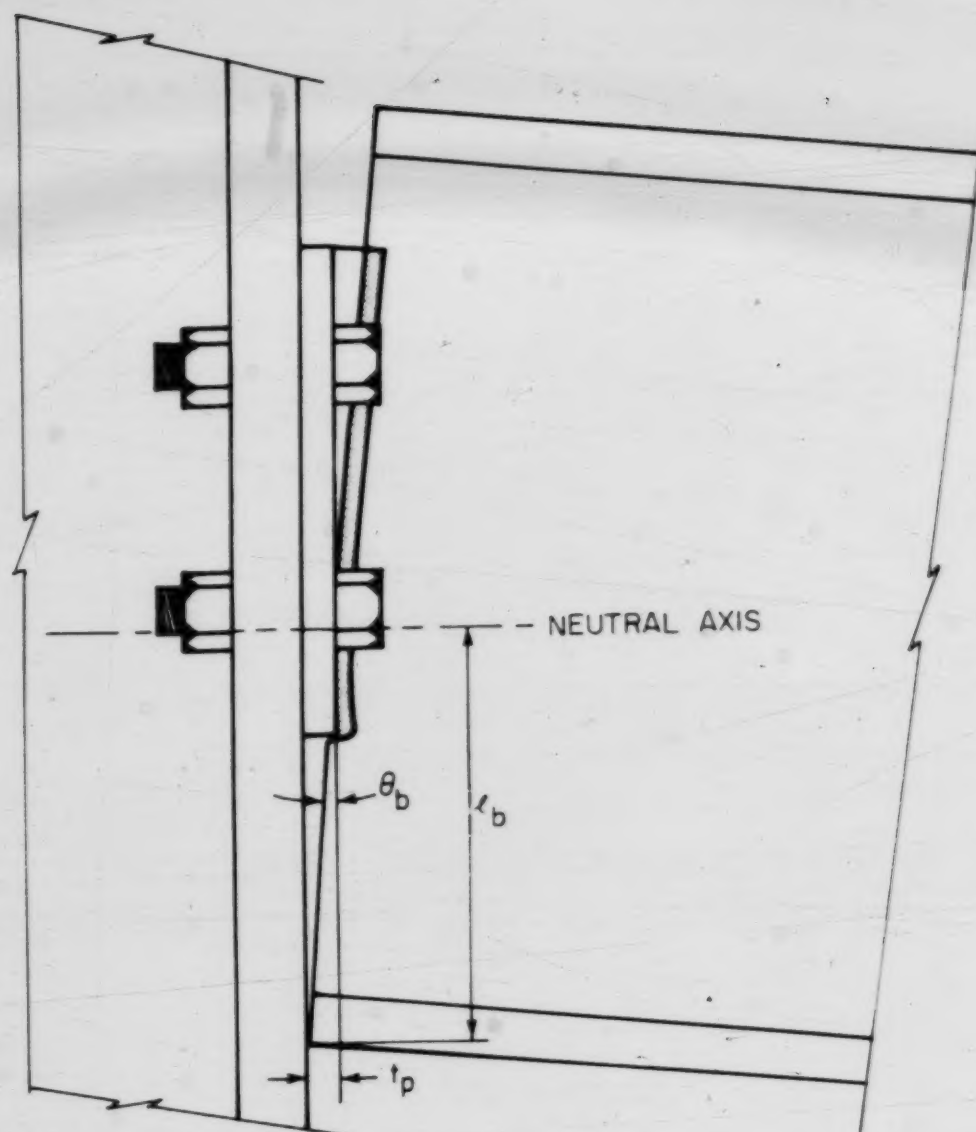


Figure 4.1 Bottom Flange Bearing of Supported Beam on Supporting Element

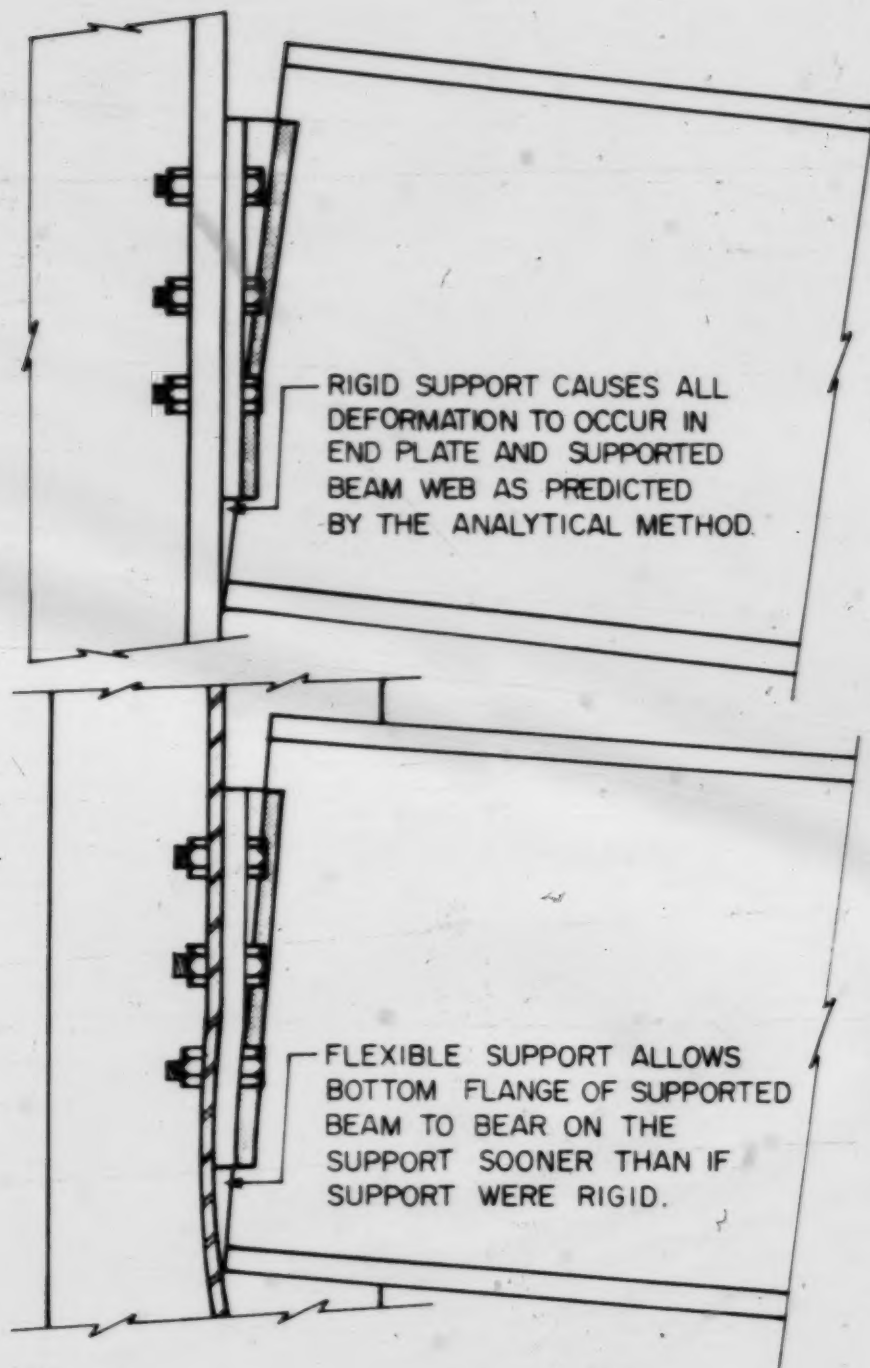


Figure 4.2 Effect of Support Stiffness on Bottom Flange Bearing

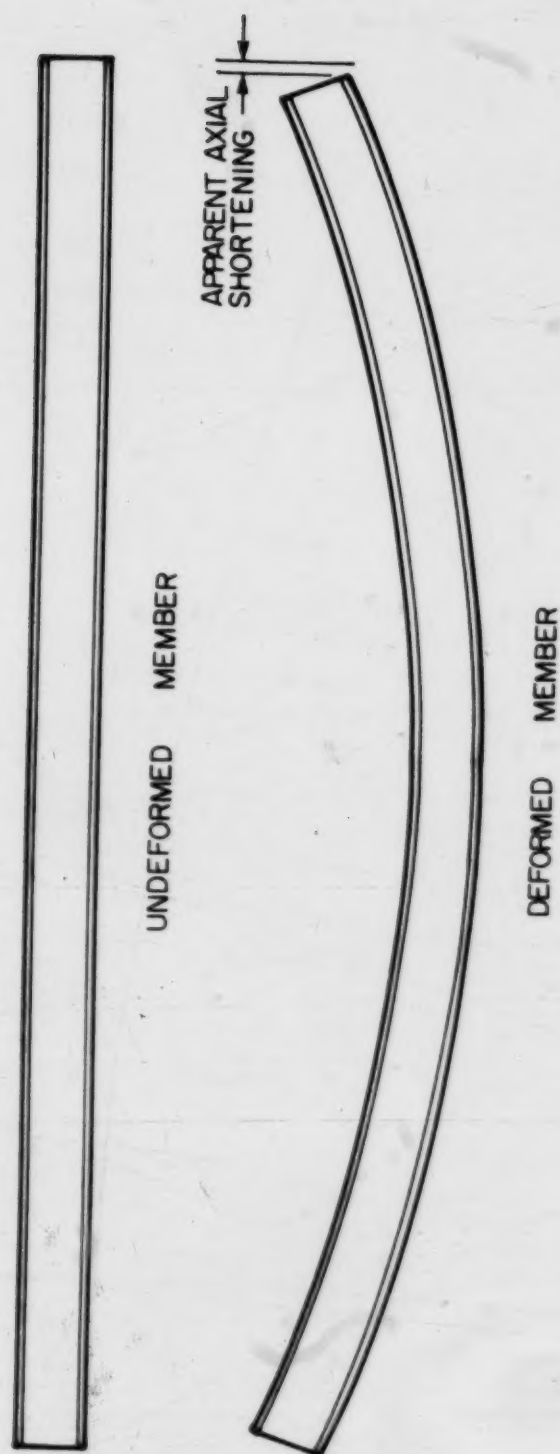


Figure 4.3 Apparent Axial Shortening of Supported Member
Under Large Transverse Deflections

5. Evaluation of the Secondary Forces that Develop in a Flexible End Plate Connection

5.1 Introduction

A prerequisite to determining the primary shear capacity of a flexible end plate connection is an analytical method to determine the magnitude and distribution of secondary forces that develop in the connection when it deforms.

The method presented in Chapter 3 to evaluate the moment-rotation relationship can be used to estimate the distribution of tensile and compressive forces that develop at the interface between the end plate and the beam web. In this chapter, consideration is given to the secondary tensile and shear forces which develop in the bolts and to the ultimate deformation levels that can be attained in flexible end plate connections prior to connection failure.

5.2 Description of Model

Although not common in flexible end plate connections, the possibility of bolt failure in deep connections exists (Sommer 1969). To estimate the reduced primary shear strength of the bolts, an analysis must consider the limiting deformation that the connection can undergo due to limited bolt strength and deformability.

To permit a rigorous analysis, the tension zone of a typical end plate shown in Figure 5.1(b) is modelled as in

Figure 5.1(a).

Four collinear horizontal elements with thickness and material properties equal to those of the end plate connection, are used to model the end plate. Hinges with rotational springs are placed where the plastic hinges were generally observed to form. For flexible end plate connections tested by Sommer (1969) and by Hafez (1982), the simultaneous formation of four plastic hinges in the plate was always observed, and is therefore assumed to be the case for this model. The width of the model is taken as one bolt hole pitch to allow a direct evaluation of secondary bolt forces at each bolt location.

Between the two inner plate elements is the very stiff junction of the connection plate and the supported beam web, represented by an undeformable element with a length equal to the thickness of the supported beam web plus two times the size of the fillet weld.

The hinges are idealized to be undeformable and capable of transferring axial and shear forces only, with flexural stiffness of the plate modelled by rotational springs at each of the hinge locations. Because the hinges at both the bolt line and at the toe of the fillet weld formed in the gross section of the plate (Hafez 1982), all four hinges are assumed to be identical with stiffness K_{R1} .

Deformation of the fillet welds and the supported beam web, assumed here to be rigid, would tend to reduce secondary forces in the connection plate and bolts, and

it is therefore conservative to neglect it.

It has been observed (Sommer 1969, Hafez 1982) that both the bolts and the adjacent region in the plate deform. The axial springs of stiffness K_{A1} , orientated perpendicular to the plate at the center line of bolt hole locations, account for the tensile stiffness of the bolts. Axial springs of stiffness K_{A2} located in the plane of the undeformed connection at the bolts represent the collective stiffnesses of the bolt in shear, the net cross-section of the plate in tension, and the bearing stiffness of the plate between the edge of the plate and the outer edge of the bolt hole. Because bolt holes are necessarily larger than the bolts, there is always the possibility of slip in a real connection. However, no consideration of this is made herein. Flexible connections do not significantly reduce the end rotation of beams assumed to be simply supported. Therefore connection rotations and deformations are calculated herein (see Chapter 6 and Chapter 8) assuming the connections to be perfect hinges. Hence, by ignoring the possibility of slip, and by calculating the rotation of all connections as if they were perfect hinges, the secondary forces predicted will always be conservatively large.

When the supporting material is thicker than the end plate, which was the case in the Sommer (1969) and Hafez (1982) tests, the bearing deformations in the end plate will predominate. Secondary forces due to bearing resistance are thereby limited by the bearing stiffness of the end plate.

Ignoring bearing deformations in the support, which can be significant if the support is as thin or thinner than the end plate, results in a conservative overestimate of the secondary horizontal forces at the bolts. This will be the adopted approach herein.

Both Kennedy et al. (1981), and Sawa et al. (1982), considered the flexural stiffness of the bolts when modelling bolted T-joints. While this flexural stiffness is not considered directly here, the shear stiffness used for describing the bolts implicitly considers it.

The entire connection plate system rests on a frictionless surface supported by a rigid base, representing the supporting member. If the base is not rigid, any deformations would tend to reduce secondary bolt forces and this assumption is therefore conservative.

To use the model, appropriate spring stiffnesses must be developed.

5.3 Analytical Relationships

Figure 5.2 shows the deformations and resulting external force system that develops when the out-of-plane deformations occur. Various free body diagrams are shown in Figure 5.3. The distributed prying force $Q(x)$ is replaced with a discrete prying force Q such that:

$$[5.1] \quad Q = \int_{x=0}^{x=x'} Q(x) dx$$

The location of the prying force resultant is taken as given in the CISC Handbook (CISC 1980), that is, either at the extreme edge of the plate or at a distance of $1.25 \times (l + f)$ measured from the centerline of the bolt hole towards the plate edge, whichever is the lesser.

From the free body diagram shown in Figure 5.3(a), the value of the prying force Q is given as:

$$[5.2] \quad Q = \frac{1}{a} \left(M + \frac{V d_h}{2 \cos \alpha_1} + \frac{H d_h}{2} \tan \alpha_1 \right)$$

If the angles α_i are assumed to be zero, (see Figure 5.2) then $\alpha_2 = \alpha_1 = \alpha$ and Equation [5.2] can be written as:

$$[5.3] \quad Q = \frac{1}{a} \left(M + \frac{V d_h}{2 \cos \alpha} + \frac{H d_h}{2} \tan \alpha \right)$$

Assuming that α_3 equals zero is tantamount to assuming that the axial deformation of the bolts, that is spring $A1$, is generally small compared to the maximum plate deformation Δ . This is generally the case, unless the connection geometry is chosen to create a very stiff connection detail. When connection proportions are selected so that α_3 is not small, that is four hinges do not form simultaneously in the end plate, then the detail is unfavourably stiff and should not be used. By using the assumption of four hinges, for stiff details the tensile forces in the bolts will be overestimated, and will in fact be greater than the bolt tensile strength. Therefore, this approach assists designers

in identifying unfavourably stiff connection details.

From the free body diagram in Figure 5.3(c) the shear force V is given as:

$$[5.4] \quad V = \frac{2M}{l} \cos \alpha$$

Combining this with Equation [5.3] gives:

$$[5.5] \quad Q = \frac{1}{a} \left(M + \frac{M d_h}{l} + \frac{H d_h}{2} \tan \alpha \right)$$

The value of M is:

$$[5.6] \quad M = K_{R1} \alpha \quad \} M_{up}$$

and the value of H is:

$$[5.7] \quad H = K_{A2} \Delta_{A2}$$

If it is assumed that the axial deformation of the end plate between the hinge locations is zero, as shown in Figure 5.2, then the deformation of spring $A2$ is related to the plate deformation by:

$$[5.8] \quad \Delta_{A2} = l (1 - \cos \alpha)$$

The assumption that all of the axial deformation occurs in axial springs $A2$ results in an overestimation of the

axial force present at any end plate deformation. This is conservative because the secondary forces are overestimated which would reduce the capacity of the connection to carry primary shear.

From the free body diagram in Figure 5.3 (a), the value of the tension force in the bolt, B, is:

$$[5.9] \quad B = Q + V \cos \alpha + P \sin \alpha$$

Using Equations [5.4], [5.5], and [3.17] to evaluate V, Q, and P respectively gives:

$$[5.10] \quad B = H \tan \alpha \left(1 + \frac{d_h}{2a} \right) + \frac{M}{a} \left(1 + \frac{d_h}{l} + \frac{2a}{l} \right)$$

5.4 Spring Stiffness Characteristics

5.4.1 Axial Spring A1

The springs with stiffness K_{A1} represent the tensile load-deformation characteristics of the bolts which, in Canada, are likely to be high-strength bolts.

From the studies done by Rumpf and Fisher (1963) on A325 bolts and by Sterling et al. (1965) on A490 bolts it is apparent that the load-deformation characteristics of high-strength bolts are primarily a function of the length of the thread in the grip and the bolt installation technique.

As the length of the threaded portion in the grip is reduced, the ultimate deformation capacities of the bolts decrease. Sterling et al. also noted that this reduction in the length of the threaded portion caused an increase in the ultimate tensile strength of the bolt. Rumpf and Fisher reported that for bolts with grip lengths up to about 200 mm, the total grip had no appreciable effect on the load deformation characteristics of the bolt beyond the proportional limit; the deformation occurring in the length of the thread within the grip governed the maximum attainable bolt elongation.

Rumpf and Fisher also noted that, for A325 bolts tightened $1/2$ to $2/3$ of a turn beyond snug, the ultimate elongation of a bolt is reduced to about 60 percent of the ultimate elongation attainable for a similar bolt installed finger-tight only. The shear strains induced in the bolt when it is tightened beyond the snug position reduce the ultimate axial strain attainable. However, as was reported by Sterling et al., the effect of torquing A490 bolts beyond snug did not appear to cause such a great reduction in the ultimate bolt elongation. For both A325 and A490 bolts, the ultimate tensile strength of the bolts was not affected by the the installation procedure.

For finger-tightened bolts, the ultimate elongation attainable ranges from about 3 to 4 mm, when the thread length in the grip is 3 mm, up to 9 mm when the thread length in the grip is 25 mm (Rumpf and Fisher). At the,

present time in Canada, bolts are to be tightened beyond snug and therefore would only have about 60 percent of these ultimate deformations, that is, 2 mm to 5 mm, if A325 or A325M bolts were used. Except for very stiff connections, this deformation is relatively small compared to the total deformation occurring at end plate connections at ultimate conditions. Therefore, neglecting bolt elongation is conservative and reduces the complexity of the model.

Furthermore, the bolts in flexible end plate connections are provided mainly to transfer beam end shear. The presence of shear strain in the bolt due to this shear force likely acts to reduce the ultimate tensile strain of the bolts even further, in a manner similar to that caused by torquing the bolts beyond snug at installation time. Therefore, it appears reasonable to neglect bolt elongation, that is, springs A1 are assumed to be infinitely stiff.

5.4.2 Axial Spring A2

The axial spring with stiffness K_{A2} represents the collective stiffness of the bolt in shear, of the reduced cross-section in tension, and of the plate between the bolt and the plate edge in bearing.

5.4.2.1 Shear Stiffness of the Bolts

The load-deformation characteristics of high-strength bolts loaded in shear have been investigated by Fisher (1965), Wallaert and Fisher (1965), Crawford and Kulak (1971), and Bahia and Martin (1980), among others. The tests

conducted by Bahia and Martin resemble most closely the characteristics of the bolts of the flexible end plate connection when they undergo horizontal deformation due to out-of-plane movement of the end plate.

The test results of interest were obtained from a series of tests on single bolt specimens loaded in single shear where the bolt threads intercepted the shear plane (which may be the case in practice). The lap plates were manufactured from plates having nominal thicknesses of 12 mm, 14 mm, 17 mm, and 20 mm. To maintain a uniform yield strength in all of the lap plates, all specimens were fabricated from the same lot of 20 mm thick plate, and then the three thinner specimen groups were obtained by machining. The mean yield strength of the plate material was reported as 272 MPa. The high strength bolts had nominal diameters of 20 mm and had a mean ultimate tensile strength of 906 MPa.

For the range of test specimens evaluated, it was concluded that the total deformation of a bolt at ultimate load is directly related to the average bearing stress that exists between the bolt and the plate. No empirical relationship was developed, however, a plot is provided which compares the average bearing stress to the total shear deformation of a bolt over the entire loading range of the test specimens. From this it is possible to develop the following expression relating average bearing stress to ultimate shear deformation:

$$[5.11] \quad \Delta_{bsmax} = k_1 \sigma_{bsmax}$$

For this particular case $k_1 = 0.0139$ when SI units are used. The coefficient k_1 is likely a function of the material properties of the connected plates, the bolt grade, the bolt diameter, and the location of the shear plane in the bolt. Investigations made by Wallaert and Fisher appear to substantiate this.

The maximum shear deformation of a bolt can also be related directly to the geometric and material properties of the bolt and the connected plates. The maximum average bearing stress that can be developed if the bolt fails in shear prior to plate failure can be expressed as:

$$[5.12] \quad \sigma_{bsmax} = \frac{V_u}{d_b t_p}$$

where the ultimate shear strength of the bolt is:

$$[5.13] \quad V_u = k_v k_{sp} A_b \sigma_{ub}$$

Substituting Equations [5.12] and [5.13] into Equation [5.11] gives:

$$[5.14] \quad \Delta_{bsmax} = k_1 k_v k_{sp} \frac{\pi d_b}{4 t_p} \sigma_{ub}$$

The coefficient k_v is a ratio of the shear strength of a bolt through the shank to its tensile strength and is usually observed to be between 0.6 and 0.7 in tests. The value of k_v observed by Bahia and Martin was 0.67. The coefficient k_{sp} is a ratio of the cross-sectional area of the bolt through the threads to the area at the shank. It normally ranges from about 0.70 to 0.80 and the value observed by Bahia and Martin was 0.78.

Equation [5.14] gives the maximum deformation of the bolt when it fails in shear. The maximum deformations, using the values of k_1 , k_v and k_{sp} obtained by Bahia and Martin, are presented in Figure 5.4 as the upper limits of a bilinear load-deformation relationship. Test results of Bahia and Martin are also plotted in Figure 5.4. In examining the test results it appeared appropriate to use a bilinear expression to model the load-deformation relationship as shown in the figure.

Up to about 80 percent of the ultimate shear capacity, the deformation-shear relationship is given by:

$$[5.15] \quad \Delta_{bs} = \frac{k_2 H}{d_b t_p}$$

where k_2 has a value equal to 0.0074 when SI units are used. It is interesting to note that the bolt deformation is inversely proportional to the plate thickness; the thicker the plate, the smaller is the bending and local bearing component of the overall deformation.

The upper portion of the bilinear expression is obtained by joining the point representing both the deformation and 80 percent of the ultimate bolt shear to that representing the maximum deformation given by Equation [5.14] and the ultimate bolt shear. The equation is:

$$[5.16] \Delta_{bs} = (5 k_1 - 4 k_2) \frac{H}{d_b t_p} - k_v k_{sp} \pi \frac{d_b}{t_p} \sigma_{ub} (k_1 - k_2)$$

As shown in Figure 5.4, the bilinear relationships correlate reasonably well with the test results. Because coefficients in Equation [5.16] have been derived empirically, it is strictly valid only for connections with material and geometric properties similar to those in the Bahia and Martin tests. It is, however, considered valid for most end plate connections of usual proportions composed of mild or medium strength steel and high-strength bolts.

Throughout the load-deformation response, the deformation is inversely proportional to the plate thickness. The thicker plates provide more restraint than do the thinner ones. Hawkins (1971) suggested that the major factor that influences the shear deformation response of a bolt is the amount of restraint that is provided by the fastened plates and that the restraint provided by the connected plates is affected greatly by the thickness of the adjoining plates as depicted in Figure 5.5

Further, most flexible end plates are between 6 mm and 10 mm in thickness. This range lies outside the range tested

by Bahia and Martin. It is assumed, because of the good inverse correlation obtained between deformation and plate thickness, that the relationship developed can be extended to the normal range of flexible end plate thicknesses.

In addition, the end plate is likely to be considerably thinner than the supporting element. In the limit, with a very thin end plate, rotational restraint of the bolt would only be provided by the thick supporting element and the bolt is bent in single curvature rather than in double curvature. To account for this the bolt deformation in each plate can be computed independently by taking one half of the deformations of each plate as calculated by using Equations [5.15] and [5.16], and adding them together.

Moreover, in flexible end plate connections the bolts resisting large horizontal shears also resist tensile forces. These tensile forces are likely to increase the bolt deformation and therefore the relationships developed not considering this increased deformation would overestimate the shear force for a given bolt deformation. The primary shear forces will have a similar effect and neglecting them would again be conservative.

5.4.2.2 Tensile Stiffness of the Net Section

Fisher (1965), in conducting tests on mild strength tension specimens with holes, observed the development of a triangular yield pattern on both sides of the hole. For specimens with a gross plate width to hole diameter ratio of from 3.0 to 7.5 the ultimate load was equal to the net

cross-sectional area multiplied by the ultimate tensile strength.

A finite element analysis done by Kato and Aoki (1970) showed, as would be expected, for the same type of specimen that the greatest strains occur near the net section. This analysis also indicated that the triangular zones of yielding around the holes, form at angles of 45 degrees to the axis of loading, as shown in Figure 5.6 (a). This zone of excessive straining persisted until the net section ruptured.

Tests conducted by Iwankiw and Schlafly (1982) on single bolt double lap joints designed to fail on the net section indicated that it is possible to reach the ultimate net section tensile capacity.

A plausible model, therefore, could consist of a series of parallel springs as shown in Figure 5.6 (b), each with a load-deformation response based on the uniaxial stress-strain characteristics of the plate material. Because of the different spring lengths, the load-deformation response for the member as a whole would be similar to the stress-strain curve but would be rounded due to the uneven straining. A simpler model consists of springs located midway between the center of the hole and the edges of the specimen with an ultimate capacity equal to the ultimate tensile strength multiplied by the net cross-sectional area.

Using the simplified model and neglecting the effect of flexural and primary shear stresses, the stress-strain curve

used to model the tensile stiffness of the net section is shown on Figure 5.7. Ultimate deformations are based on strains at the edge of the hole which are proportional to those of the model springs.

Equations describing the various linear portions of the curve shown in Figure 5.7 are:

$$[5.17] \quad H = \frac{2 E_p}{p} (p - d_h) t_p \Delta_{ns}$$

$$\text{when} \quad 0.0 < \Delta_{ns} < \frac{\sigma_{yp} p}{2 E_p}$$

$$[5.18] \quad H = (p - d_h) t_p \sigma_{yp}$$

$$\text{when} \quad \frac{\sigma_{yp} p}{2 E_p} < \Delta_{ns} < \frac{5 \sigma_{yp} p}{E_p}$$

$$[5.19] \quad H = (p - d_h) t_p \left[20 \left(\frac{\sigma_{up} - \sigma_{yp}}{1 - 100 \epsilon_y} \right) \frac{\Delta_{ns}}{p} + \left(\frac{\sigma_{yp} - 100 \epsilon_y \sigma_{up}}{1 - 100 \epsilon_y} \right) \right]$$

$$\text{when} \quad \frac{5 \sigma_{yp} p}{E_p} < \Delta_{ns} < \frac{p}{20}$$

$$[5.20] \quad H = (p - d_h) t_p \sigma_{up}$$

$$\text{when} \quad \frac{p}{20} < \Delta_{ns} < \frac{p}{20}$$

$$[5.21] \quad H = (p - d_h) t_p (20 (\sigma_{yp} - \sigma_{up}) \frac{\Delta_{ns}}{p} + 3\sigma_{up} - 2\sigma_{yp})$$

$$\text{when } \frac{p}{10} < \Delta_{ns} < \frac{p}{4}$$

5.4.2.3 Bearing Stiffness of the Plate

de Back and de Jong (1968) observed in single bolt, double shear lap splice tests of mild steel specimens that for plates having gross width-to-loaded edge distance ratios greater than or equal to about 1.60, failure occurred in a bearing or tear-out fashion. As the bolt pitch-to-edge distance ratio for end plate connections falls within the range of 1.8 to 3.3, bearing deformations are likely to be an important consideration.

The results of the test series of Owens et al. (1981) performed on single bolt, single shear lap plate splices with 20 mm diameter high strength bolts in 6 mm thick by 72 mm wide lap plates with a 22 mm diameter hole centered in the plate located 40 mm from the loaded edge replicate well the characteristics of end plates. The steel was Grade 43 with a nominal yield strength of about 300 MPa.

Based on the non-dimensionalized experimentally obtained bearing stress vs. deformation curves of Owens et al., the approximate bilinear relationships of

$$[5.22] \quad H = \sigma_{up} t_p d_b \Delta_{be}$$

when $0 < \Delta_{be} < \frac{e}{d_h}$

and

$$[5.23] \quad H = \sigma_{up} t_p e$$

when $\frac{e}{d_h} < \Delta_{be} < 0.40 d_h$

are proposed. The upper limit is assigned because bearing rupture occurred at a deformation level, $\Delta_{be\max}$, of about 40 percent of the bolt diameter.

Equations [5.22] and [5.23] and experimental data of Owens et al. are shown in Figure 5.8. Thomas and Bennetts (1982), Chung Wing and Harris (1983), and CAN3-S16.1-M78 (CSA 1978) all use Equation [5.23] to predict the ultimate bearing strength.

The load-deformation response of specimens tested by Chung Wing and Harris was similar to that observed by Owens et al., with the exception that the connections appeared stiffer at load levels below that causing a bearing stress of σ_{up} . This response is unexpected as the loaded edge distance to bolt hole diameter ratio of the tests of Chung Wing and Harris were about the same as those of Owens et al.. It is attributed to the fact that the bolts were installed by Chung Wing and Harris to 1/2 of a turn past snug, whereas Owens et al. installed bolts to finger tightness only.

Frank and Yura (1981) observed that as lap plate width increases, all other details remaining unchanged, the

ultimate bearing deformation of the specimens increase with no significant change in ultimate strength. This greater ultimate deformation was also observed by Thomas and Bennetts (1982) for one test with a large plate width-to-bolt hole diameter ratio. The Chung Wing and Harris tests, having relatively large plate width to bolt hole diameter ratios, also appeared to have greater ultimate deformations than the tests of Owens et al.. Although none of these researchers discussed the issue, it appears that the ultimate deformability of bearing specimens is a function of the gross plate width to bolt hole diameter ratio.

Richard et al. (1980) tested single high-strength bolt single shear lap plate specimens of mild steel. They report initial stiffnesses greater than that reported by Owens et al.. These greater stiffnesses are attributed to the clamping force of the pretensioned bolts. As in flexible end plate connections, the bolt tensions that occur with end plate distortion will reduce the clamping force, and so the model based on the Owens et al. tests is considered valid.

5.4.2.4 Application

The axial spring with stiffness K_{A2} has a load-deformation response composed of three parts. For a given horizontal force H , the resulting spring deformation Δ_{A2} is:

$$[5.24] \quad \Delta_{A2} = \Delta_{bs} + \Delta_{ns} + \Delta_{be}$$

Because the magnitudes of the three component deformations of Δ_{A2} are not known initially, an iterative procedure using a computer or a programmable calculator is used to determine the deformations corresponding to the force H .

5.4.3 Rotational Spring R_1

Rotational springs with stiffness K_{R1} to account for the flexural stiffness of the end plate are located where plastic hinges formed in T-section tests (Hafez 1982). The hinge rotation α as a function of the plate deformation and geometry is:

$$[5.25] \quad \alpha = \sin^{-1} \frac{\Delta}{l}$$

For elastic behaviour⁶, the plate deflection in terms of the moment M at the hinges is:

$$[5.26] \quad \Delta = \frac{2 M l^2}{E_p p t_p^3}$$

Combining Equations [5.25] and [5.26] gives:

$$[5.27] \quad M = \frac{p t_p^3 E_p}{2 l} \sin \alpha$$

Assuming a perfectly elasto-plastic moment-curvature relationship and that the maximum moment obtainable is

$M_{up} = pt_p^2 \sigma_{up} / 4$, then the limiting value of the angle α for elastic behaviour is:

$$[5.28] \quad \alpha_y = \sin^{-1} \left(\frac{\sigma_{up}}{2 E_p} \frac{l}{t_p} \right)$$

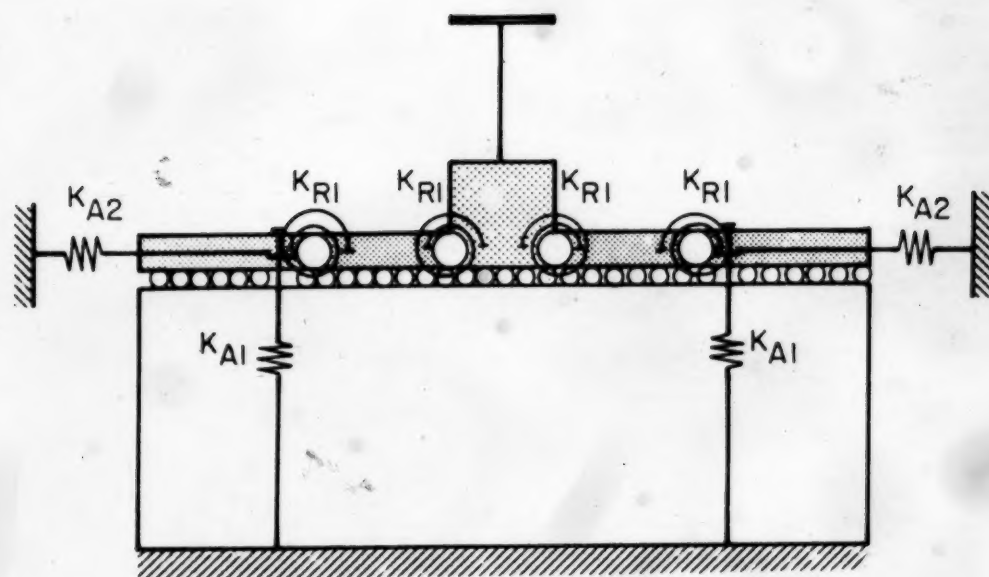
In the inelastic region, as the deflection Δ becomes significant, it is necessary to consider the reduction in the moment developed due to the presence of axial load as given by Equation [3.7]. Combining Equations [3.7] (with $P_{up} = P_{yp}$), and [3.17] (ignoring the flexural term), the M - α relationship is:

$$[5.29] \quad M = M_{up} \left(1 - \frac{H^2}{\cos^2 \alpha P_{yp}} \right)$$

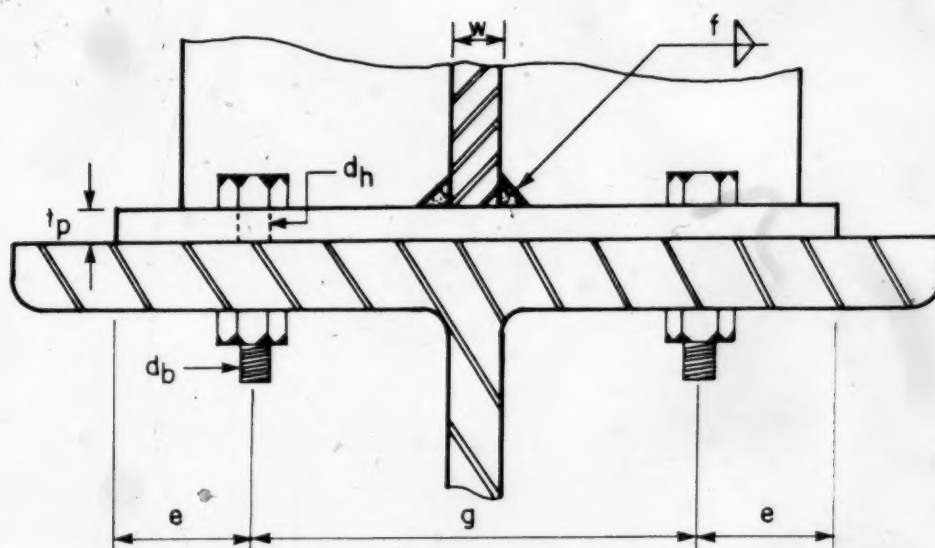
Table 5.1 shows that for the 27 T-section tests performed by Hafez (1982), when 6.4 mm (1/4 inch) plate is used the maximum value of α was 0.25 radians on the average, and when 9.5 mm (3/8 inch) and 12.7 (1/2 inch) plate was used it was about 0.45 radians. These values are used as ultimate deformations in the analysis to follow.

Table 5.1 Ultimate Rotation of Plastic Hinges in End Plate
Specimens, Hafez (1982)

Test No.	Plate Thickness (inches)	Bolt gage (inches)	Rupture Angle (rads)	Average Rupture Angle (rads)
11 12 14 26 27	1/4	4	0.222 0.295 0.251 0.200 0.210	0.236
1 4 15 22 23	1/4	5.5	0.282 0.270 0.250 0.231 0.247	0.256
8 9 10 13 24 25	3/8	4	0.475 0.438 0.330 0.498 0.495 0.445	0.447
3 6 7 16 20 21	3/8	5.5	0.374 0.480 0.500 0.500 0.500 0.495	0.475
2 5 17 18 19	1/2	5.5	0.468 0.440 0.440 0.350 0.455	0.431



a)



b)

Figure 5.1 Model of Flexible End Plate Connection

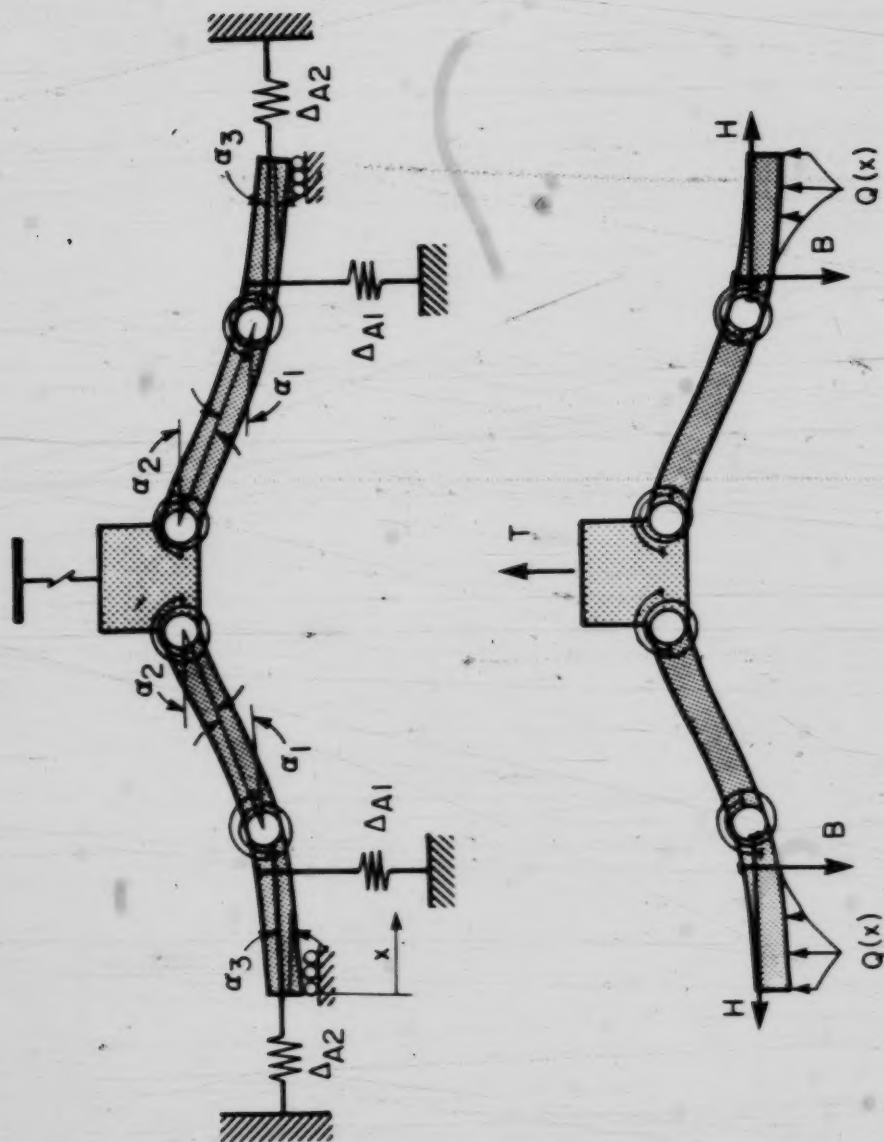


Figure 5.2 Model of Flexible End Plate Connection In Deformed Position

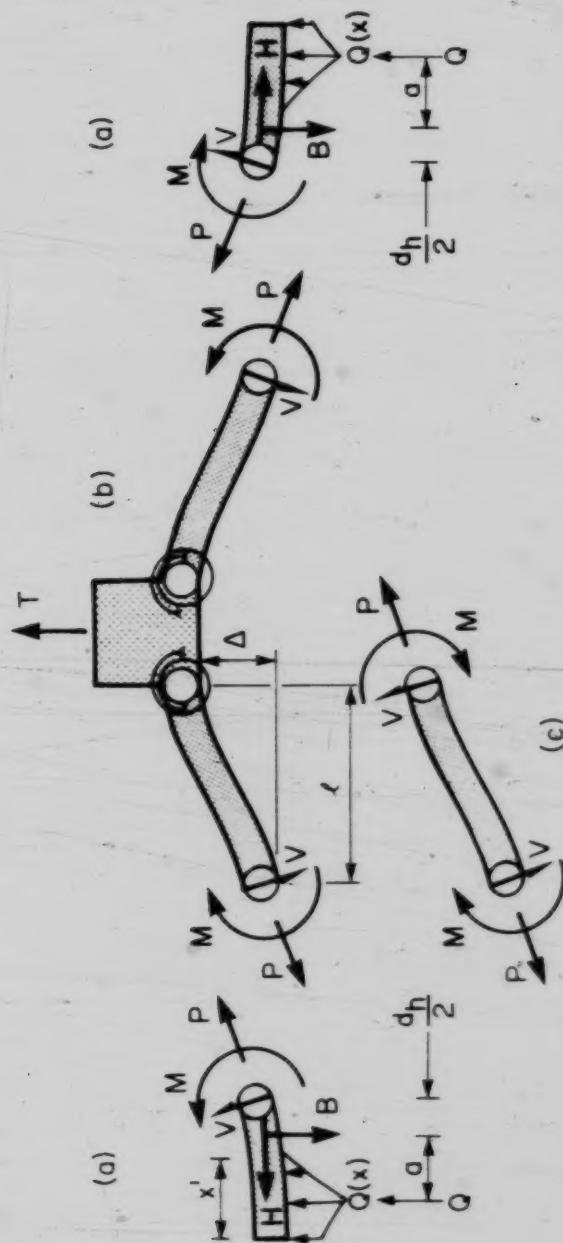


Figure 5.3 Free Body Diagrams of Flexible End Plate Connection Model

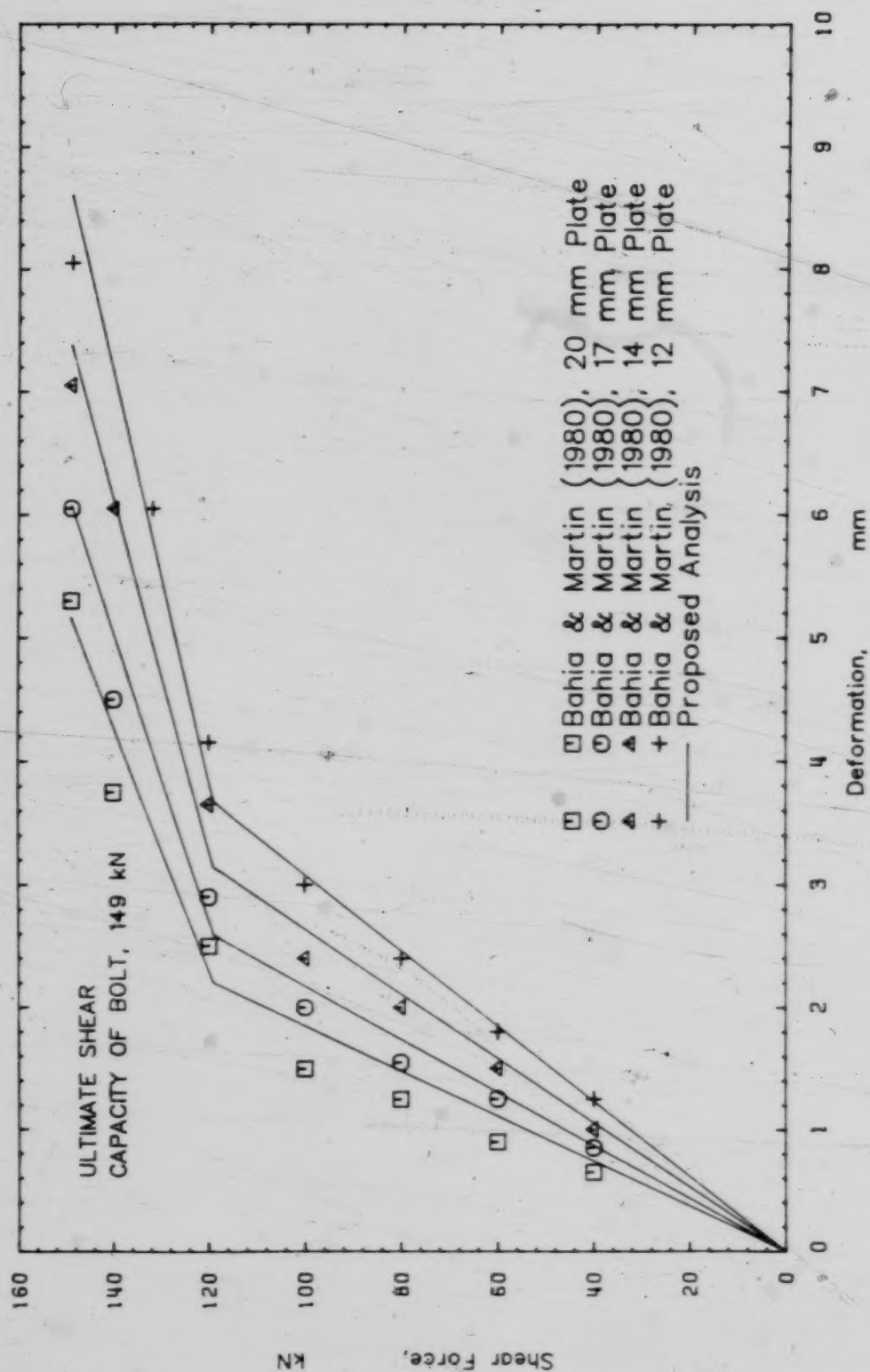


Figure 5.4 Load-Deformation Relationship of a High-Strength Bolt Loaded in Single Shear With Threads in Shear Plane

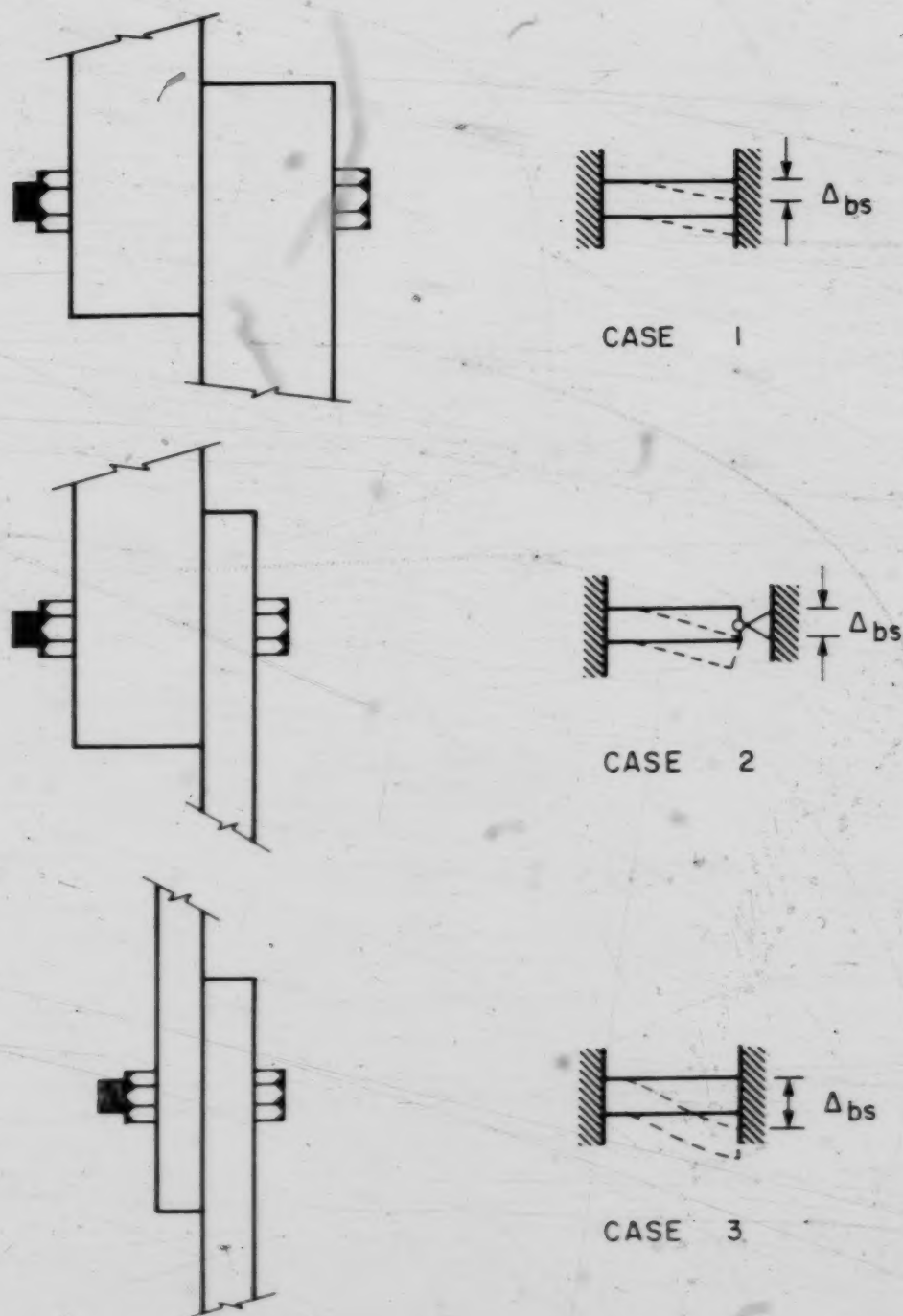


Figure 5.5 Effect of Lap Plate Thickness on Shear
Deformation Characteristics of High-Strength Bolts

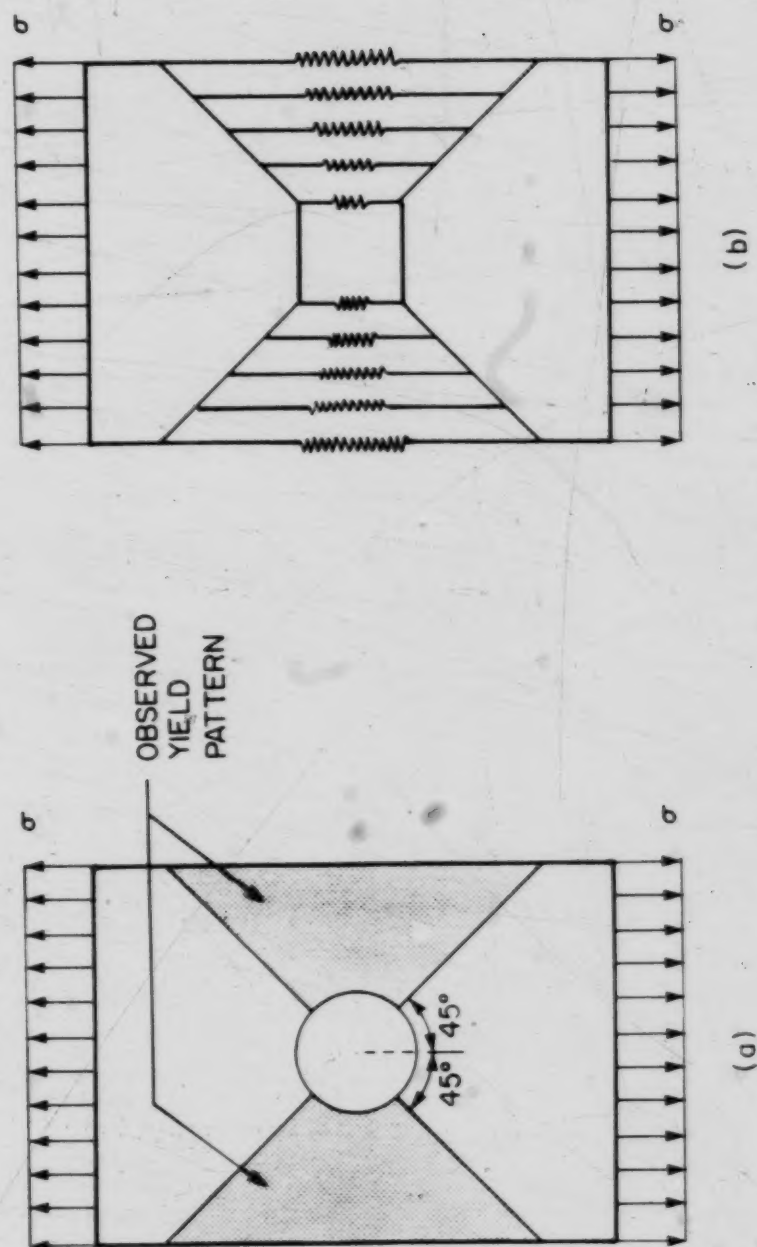


Figure 5.6 Model of a Tension Specimen With a Hole

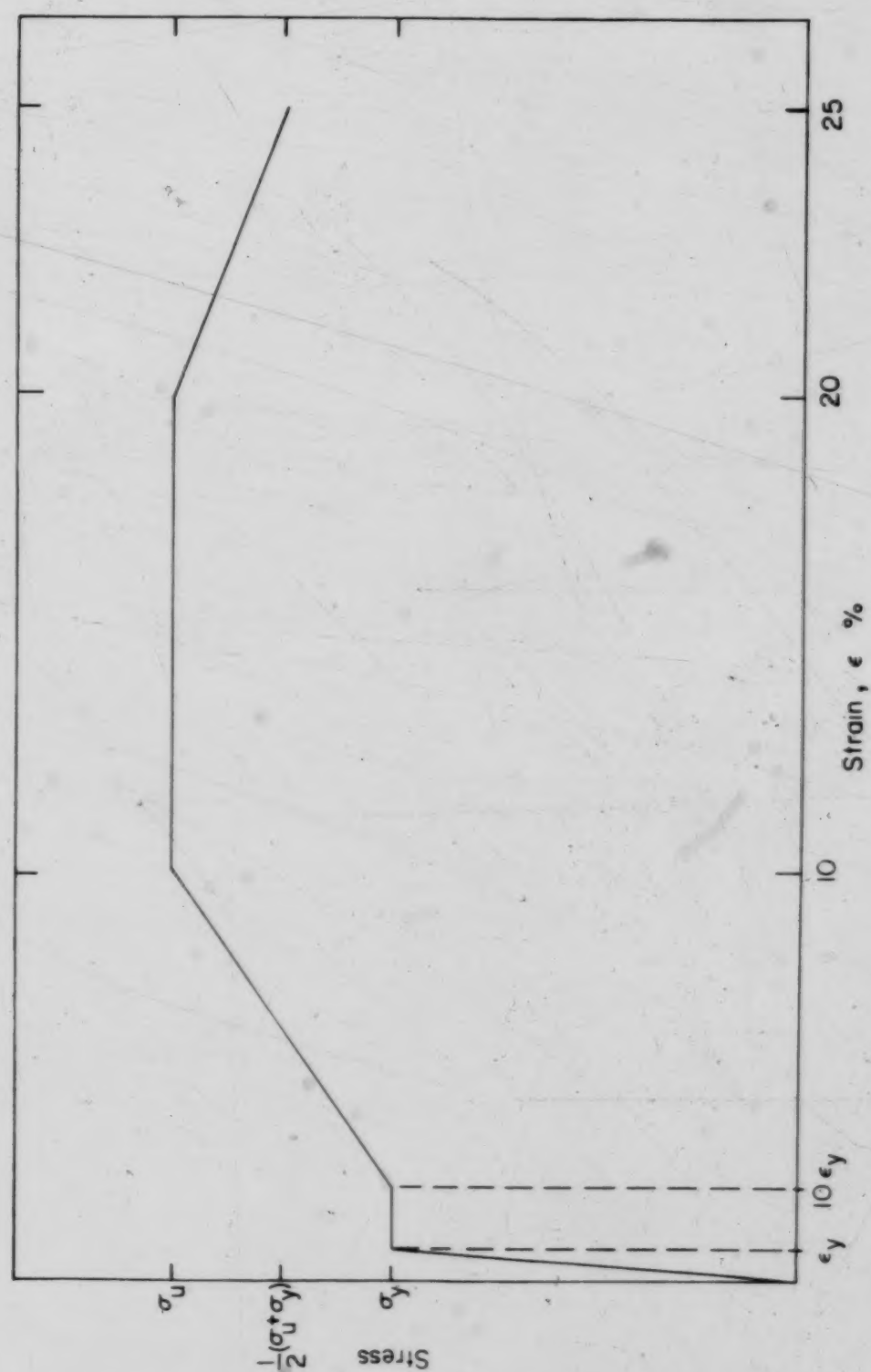


Figure 5.7 Idealized Stress-Strain Curve for Flexible End Plate Material

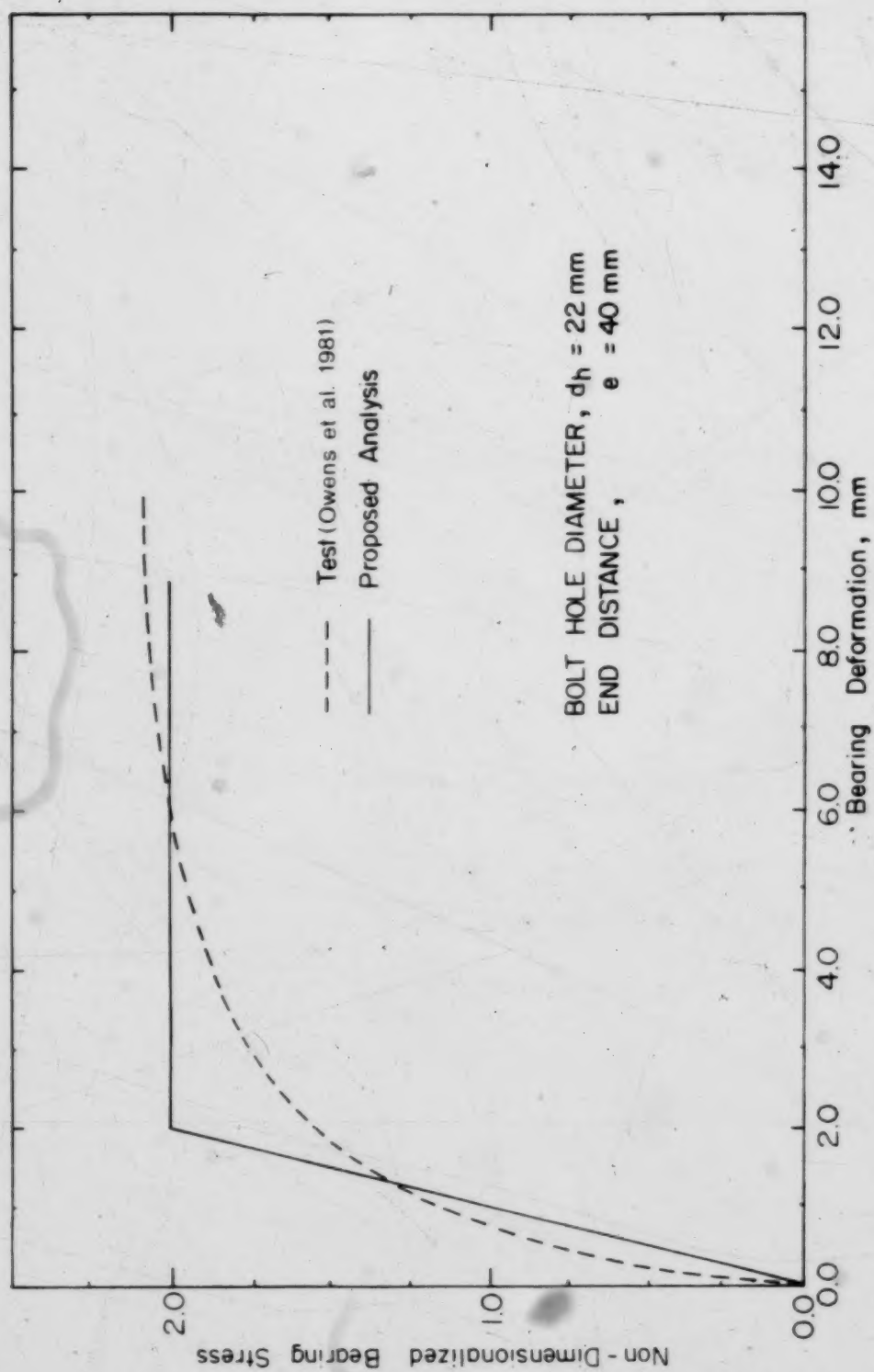


Figure 5.8 Idealized Bearing Load-Deformation Response for Flexible End Plate at Bolts

6. Evaluation of the Primary Shear Capacity of a Flexible End Plate Connection

6.1 Introduction

The current design procedure for end connections assumed to transmit shear only is to design the components of the connection only for this shear. The secondary forces that develop in the connection due to its distortion will, however, reduce the capacity of the components to carry the primary shear, with the result that the margins of safety may be less than those implied in the governing design standards. By evaluating the secondary forces at the factored load levels (a procedure consistent with Limit States Design) it is possible to determine the degradation in primary shear capacity of each of the components and therefore, the factored shear resistance of the connection. This factored shear resistance must be equal to or greater than the factored shear.

Figure 2.4 presents a flow chart for the design process of a flexible end plate connection. For a beam of a given size, length, and loading configuration, the factored end shears are computed assuming the beam to be simply supported. The end rotation is calculated under the same assumption, recognizing that the actual end rotation at the factored load level will be slightly less because of the end moments developed. The connection is sized for the factored shear, neglecting at this stage the secondary force effects.

Using the model developed in Chapter 5, the secondary forces in the various elements of the proposed connection can be established for the end rotation just calculated. In this chapter, methods are given for estimating the capacity of the elements to carry primary shear in the presence of the secondary forces. If the shear resistance of the critical element is less than the factored shear, the size of the connection is increased and the new connection is analyzed.

The general approach taken to arrive at the total primary factored shear resistance of each of the critical elements, e. g. the bolt group, is as follows:

1. compute the secondary forces throughout the depth of the connection,
2. determine the primary shear force that can be carried in conjunction with the secondary forces as a function of depth,
3. the total primary shear is the integral of the primary shears over the depth.

The validity of this lower bound approach depends upon the elements having sufficient ductility so that the ultimate capacity can be integrated over the depth of the connection.

Since the secondary forces to be used are computed for factored level loads, no further factoring is required. It is always possible that some elements of the connection

assembly may have capacities greater than the nominal values. The secondary forces that would be developed in such elements would, of course, also be greater than those corresponding to nominal capacities. It is considered that this latter possibility is adequately covered by using factored resistances for the strengths of the connection components.

6.2 Bolts

Above the neutral axis of the connection the bolts are subject to horizontal shears and tensions as well as primary shears. The tension and horizontal shear forces increase with distance from the neutral axis. The factored resistance of bolts subject to shear and tension, based on the work of Chesson et al. (1965) and as given in CSA Standard CAN3-S16.1-M78 (CSA 1978) is:

$$[6.1] \quad V_f^2 + \beta T_f^2 \leq 0.56 \phi^2 \beta (A_b \sigma_{ub})^2$$

The factored shear force, V_f , is the vector sum of the vertical shear and the horizontal shear, H . Considering this and using Equation [6.1], the shear that can be carried by the pair of bolts at the i th level is calculated by:

$$[6.2] \quad V_i = 2 \sqrt{[(0.75 \phi A_b \sigma_{ub})^2 - B_i^2] \beta - H_i^2}$$

in which the factored tensile force T_f is represented by the

tensile force in the bolts, B_i , arising from the secondary forces in the connection.

The capacity of the bolts may be limited to the bearing capacity of the end plate and therefore the shear at the i th level for two bolts should not exceed:

$$[6.3] \quad V_i = 2 \phi t_p e \sigma_{up} \leq 6 \phi t_p d_b \sigma_{up}$$

based on the bearing resistance expression given in CAN3-S16.1-M78.

Equation [6.3] should also be used to check the bearing capacity of the bolts on the supporting member using, of course, the appropriate geometric and material properties.

Assuming that sufficient ductility exists, the shear strength of the entire bolt group is given by:

$$[6.4] \quad V_{rb} = \sum_{i=1}^{i=n} V_i$$

When the shear strength at a given level is evaluated by Equation [6.2], in some instances it will be found to be zero. This indicates that the capacity of the bolts at that level has been exhausted by the secondary forces alone. Because this condition indicates that the connection is unfavourably stiff, it appears reasonable to reject such a connection.

The formulation for the primary shear resistance of the bolts does not take into consideration the frictional force

that could be developed on the faying surface between the end plate and supporting member due to the compressive force acting on the bottom portion of the connection. For those connections carrying large shears, where the frictional force would have the maximum benefit, calculations indicate that the shear carried by the frictional component is only about 5 to 10 percent of the factored shear.

6.3 Plate

The three modes of failure to be considered for the end plate subject to in-plane shear forces are: maximum shear capacity, excessive deformation, and shear buckling.

Even neglecting post buckling strength with the formation of a tension field and considering the greatest slenderness ratios likely to be encountered, the shear buckling mode will not be critical. For example, for a plate thickness of 6 mm and a 150 mm bolt gage, the critical elastic shear buckling stress is some twenty times the tensile yield stress. This mode need not, therefore, be considered further.

Eyre (1973) extended the earlier work of Hall and Newmark (1957), and investigated the influence of shear deformation on the total deformation of steel beams. He established the non-dimensionalized shear stress-strain relationships shown on Figure 6.1. (The empirically derived parameters shown in Figure 6.1 are those for SI units.) The maximum shear stress obtained is about 1.4 times the tensile

yield strength of the material, or slightly more than two times the value specified in CSA Standard CAN3-S16.1-M78 (CSA 1978) for webs of stocky beams. With the shear carried equally by the plate extending from both sides of the supported member web, and considering the critical section to be that through the bolt holes, the shear resistance of the end plate, neglecting the secondary forces developed in the plate, is:

$$[6.5] \quad V_{rp} = 2 \phi (h - n d_h) t_p 1.4 \sigma_{yp}$$

The Commentary on CSA Standard CAN3-S16.1-M78 (CISC 1980) indicates that the limit on shear stress of 0.66 times the tensile yield strength is more to prevent excessive deformation than to prevent rupture. Based on Eyre's (1973) equations, setting a shear deformation limit in the plate of 1 mm for the minimum pitch of 3 bolt diameters the net section is 67 percent of the gross section and the average maximum shear stress is about 80 percent of the tensile yield strength. This is about 19 percent greater than the average maximum shear stress specified in CAN3-S16.1-M78. The corresponding shear stress on the gross section is about 0.55 times the tensile yield stress, giving a shear resistance at this limiting deformation of:

$$[6.6] \quad V_{rp} = 2 \phi h t_p 0.55 \sigma_{yp}$$

when considering the two halves of the end plate.

Equation [6.6] does not recognize that in the upper part of the connection, above the neutral axis, the deformation generates horizontal forces H_i at the i th level of each pair of bolts. Using the Huber-von Mises yield criterion and taking the comparative stress to be equal to the ultimate tensile strength of the plate, the effect of these forces on the vertical shear capacity can be considered. For a depth of connection equal to the bolt pitch giving an area for the two sides of the plate of $2t_p(p - d_h)$, the shear carried at the i th level is:

$$[6.7] \quad V_i = 2 t_p (p - d_h) \sqrt{\frac{1}{3} (\phi^2 \sigma_{up}^2 - \frac{H_i^2}{(p - d_h)^2 t_p^2}}$$

However, the value given by Equation [6.6] should not be exceeded and will control the lower levels of the connection. When the H_i forces are taken into account, the reduced total shear resistance is the sum of the V_i values:

$$[6.8] \quad V_{rp} = \sum_{i=1}^{i=n} V_i$$

This analysis does not directly take into account the effect of the out-of-plane flexural deformations at the toe of the fillet welds and on a line at the inner edge of the bolt holes. It is recognized that significant straining occurs at these locations and in section 5.4.3 a deformation

limit was imposed to preclude the rupture that may result. Thus, the ultimate tensile strength used as the comparative stress for the Huber-von Mises criterion is not exceeded.

Equations [6.6] and [6.7] are based on a shear rupture through the net section, a mode of failure not observed in the tests of Sommer (1969) and Hafez (1982), where failures generally occurred because of excessive out-of-plane deformations.

6.4 Fillet Welds

The capacity of the fillet welds to carry primary shear must be evaluated separately above and below the neutral axis. In addition to primary shear, the fillet welds above the neutral axis must carry tension T , as shown in Figure 6.2. Provided that all components have sufficient ductility, a lower bound solution can be obtained by assigning the horizontal forces H and the moments M to the end plate, leaving the welds to carry the tension and shear. If, after considering the secondary forces, the welds do not have sufficient capacity to resist the vertical shear, the weld size must be increased to safeguard against rupture.

The procedure for determining the capacity of the fillet welds to carry primary shear in the tension zone is similar to that used for the bolts, that is:

1. at any level, the tensile forces are computed for the deformation that exists,

2. using some interaction equation, the primary shear capacity at any level is determined by relating the capacity of the fillet welds to carry transverse shear due to tension, to longitudinal shear due to the primary shear force, and
3. summing the primary shears throughout the tension zone gives the total factored shear that the weld can carry above the neutral axis.

The approach currently specified in CSA Standard CAN3-S16.1-M78 (CSA 1978) for fillet welds loaded by tension and shear, indicates that the transverse shear strength of a weld is equal to the longitudinal strength of that weld and that the two shear stress components should be added vectorially. This is equivalent to saying that the interaction diagram is circular. Butler and Kulak (1971) investigated fillet welds loaded with transverse and longitudinal shear components and showed that the strength of the welds varied with the angle of loading. For example, welds loaded only by transverse forces were about 45 percent stronger than those loaded only by longitudinal forces. The strength established by Butler and Kulak has been incorporated in the CISC design tables for eccentrically loaded welds (CISC 1980). Kennedy and Kriviak (1985) evaluated the experimental results of Kato and Morita (1969), Butler and Kulak (1971), and Clark (1971) and proposed the parabolic interaction equation:

$$[6.9] \quad 1.2 \left(\frac{V_T}{V_U} \right)^2 - \left(\frac{V_T}{V_U} \right) + \left(\frac{V_L}{V_U} \right) = 1.0$$

The value of the ultimate shear, V_U , for shear through the throat of the weld as given in CAN3-S16.1-M84 (CSA 1984) is:

$$[6.10] \quad V_U = 0.67 \phi A_w X_u$$

The coefficient 0.67 used in this expression appears to be quite conservative. Higgins and Preece (1969) note values of 0.85 to 0.90, except in the case of high strength steels and electrodes where they observed a value of about 0.70. Butler and Kulak (1971) reported 1.0; Dawe and Kulak (1972) reported 0.90; and Swannell (1981) reported 1.0. Kato and Morita (1969) give a value of 0.66, the only result consistent with CAN3-S16.1-M84. A statistical evaluation of these and further test results may show that a larger coefficient can be used while still maintaining the required reliability index.

Kennedy and Kriviak (1985) indicate that shear at the fusion face does not govern the fillet weld strength for commonly used combinations of base metal and electrodes. In Equation [6.9], when dealing with a depth of plate equal to the bolt pitch at the i th level, the longitudinal shear, V_L , is the i th level primary shear force V_i , and the tensile force, V_T , is the tensile force T_i at the i th level. Making

these substitutions and rearranging Equation [6.9] gives:

$$[6.11] \quad v_i = 1.34 + p \frac{f}{\sqrt{2}} x_u + T_i - \frac{1.2 T_i^2}{0.67 + A_w x_u}$$

where the throat area of the two fillet welds for one bolt pitch is:

$$[6.12] \quad A_w = \frac{2}{\sqrt{2}} p f$$

For a lower bound solution, the compressive force below the neutral axis can be considered to be transmitted directly into the web of the supported member. The fillet welds are therefore available to carry primary shear calculated using Equation [6.10] where A_w is:

$$[6.13] \quad A_w = \frac{2}{\sqrt{2}} f l_{bna}$$

This approach satisfies the three lower bound solution requirements in that all the internal and external forces are in equilibrium, the internal forces assumed in no case exceed the relevant force capacity, and ductile behaviour in compression is assured as is noted in both the compression T-tests and the full scale connections of Hafez (1982).

The total shear resistance of the fillet welds then is:

$$[6.14] \quad v_{rw} = \sum_{i=1}^{i=n-j} v_i + 0.67 + \frac{2}{\sqrt{2}} f l_{bna} x_u$$

6.5 Beam Web

Because the end plate does not extend over the full depth of the supported member web, the shear capacity of the beam web at the connection will be less than that of the full web. The connection of the beam to the supporting member may also require coping of either or both the top or bottom flanges of the beam. Thus the shear strength of the web must be determined for uncoped members, top flange coped members, and top and bottom flange coped members.

Consider the uncoped beam shown in Figure 6.3. In addition to the shear transferred to the web by the fillet welds, the web at the horizontal limit of the welds must carry the tensile and compressive forces generated by the deformation of the end plate connection. Consider first the web above the neutral axis. The tensile stresses at each and every level due to the connection deformation are known. The capacity of the web to carry shear stresses can be determined using some yield criterion such as the Huber-von Mises criterion. Because the ultimate shear strength in beam webs is taken to be $0.66\sigma_{yw}$, as given by CAN3-S16.1-M78, rather than the Huber-von Mises shear yield value of $\sigma_{yw}/\sqrt{3}$, the comparative stress using the yield criterion can be increased by $0.66\sqrt{3} = 1.14$ times.

For a member subjected to uniaxial tension and shear, the Huber-von Mises criterion with the above adjustment is:

$$[6.15] \quad \tau = \sqrt{\frac{1}{3} [(1.14 \sigma_{yw})^2 - \sigma^2]}$$

For an elemental length of dy , the primary shear resistance is:

$$[6.16] \quad V_i = w \, dy \, \sqrt{\frac{1}{3} [(1.14 \phi \sigma_{yw})^2 - (\frac{T_i}{w \, dy})^2]}$$

As shown in Figure 6.4, column action is developed above the top of the fillet welds in the short section of web. This is due to the upward acting shear. The force developed in this portion is limited to the shear stress of $0.66 \sigma_{yw}$, resulting in a shear resistance of:

$$[6.17] \quad V_c = 0.66 \phi \, d_t \, w \, \sigma_{yw}$$

but is not greater than the yield stress on the area of the web enclosed by the welds, that is:

$$[6.18] \quad V_c = \phi \, f \, w \, \sigma_{yw}$$

Below the neutral axis, the web subject to compressive stresses acts in a manner similar to the portion above. The shear capacity for an elemental length is as given by Equation [6.16]. The total shear capacity of the web, taking into account the forces due to the deformation of the connection, is then the sum of the three portions described above. The equation for this is as follows:

$$[6.19] \quad V_{rm} = \sum_{i=1}^{i=\frac{h}{d}} V_i + 0.66 \phi d_t w \sigma_{yw} < \sum_{i=1}^{i=\frac{h}{d}} V_i + \phi f_w \sigma_{yw}$$

where the summation is over the full depth of the end plate.

For beams with top flange copes, Equation [6.19] would apply except that the term for column action equals zero. Furthermore, the possibility of instability of the web should be investigated (see Cheng (1984)). At the end of the cope, as shown in Figure 6.5, the interaction of shear and moment should be considered. The ASCE publication "Plastic Design in Steel" (ASCE 1971) gives a relationship for rectangular sections which could conservatively be applied here, with the assumption that the bottom flange does not exist. This relationship is:

$$[6.20] \quad \frac{M_f}{M_{rc}} + \left(\frac{V_{ft}}{V_{rc}} \right)^4 = 1.0$$

When both the bottom and top flanges are coped the relationships developed for members with only top flanges coped are applicable as well.

6.6 Other Considerations

It is necessary that the supporting member be adequate to carry the primary and secondary forces delivered to it by the connection. Therefore, in addition to the evaluation of the shear capacity, the effect of end moment on the supporting member should also be evaluated.

Where beams with end plate connections frame into spandrel girders it would seem reasonable to use as flexible an end plate connection as possible in order to limit the amount of torsional moment that can be developed in the girder. In any event, it is recognized that the twisting is self-limiting.

When flexible end plate connections are used to support laterally unsupported beams it is important that the torsional stiffness of the connection be proven to be adequate. Bennetts et al. (1982) have shown that the torsional stiffness can be relatively small, as would be the case for end plates of small depth. The amount of twist would, of course, be primarily dependent upon the flexibility of the portions of the web not connected to the end plate. Milek (1980) warns that the torsional stiffness of beams and girders with copes is further reduced from that of uncoped members supported by flexible end plates.

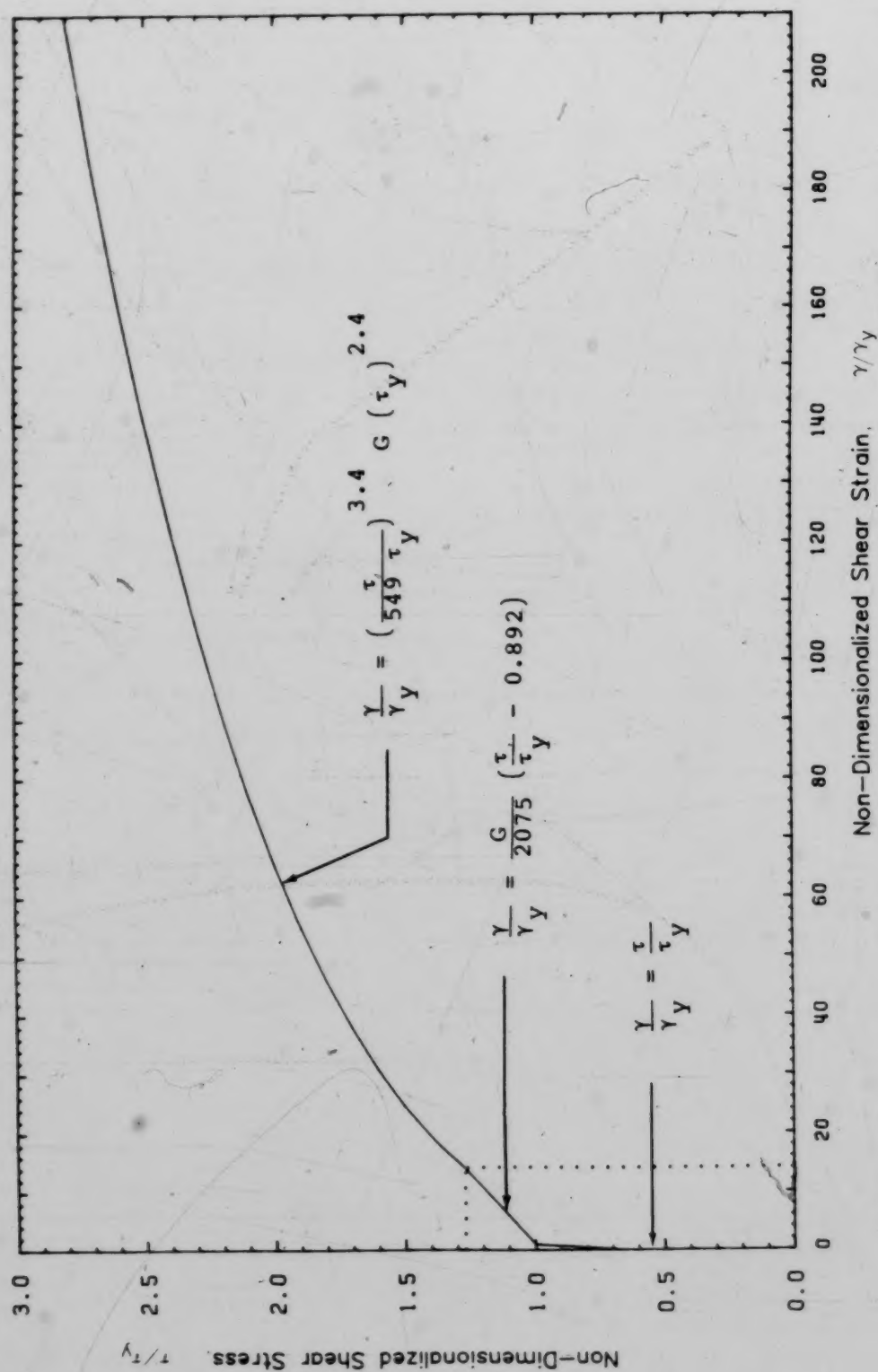


Figure 6.1 Shear Stress-Strain Relationship (Eyre, 1973)

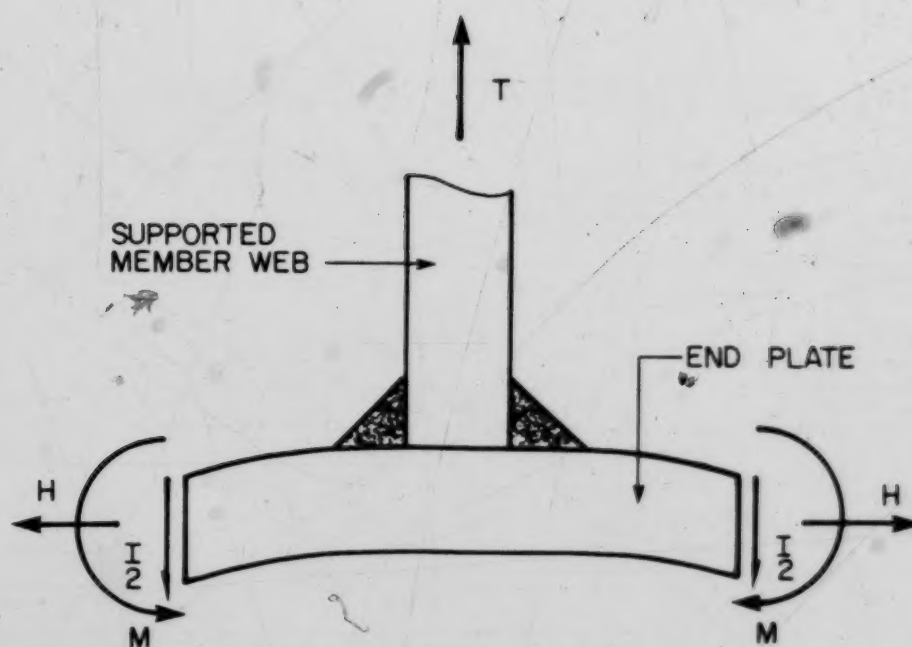


Figure 6.2 Secondary Forces Near the Fillet Welds in the Tension Region of and End Plate Connection

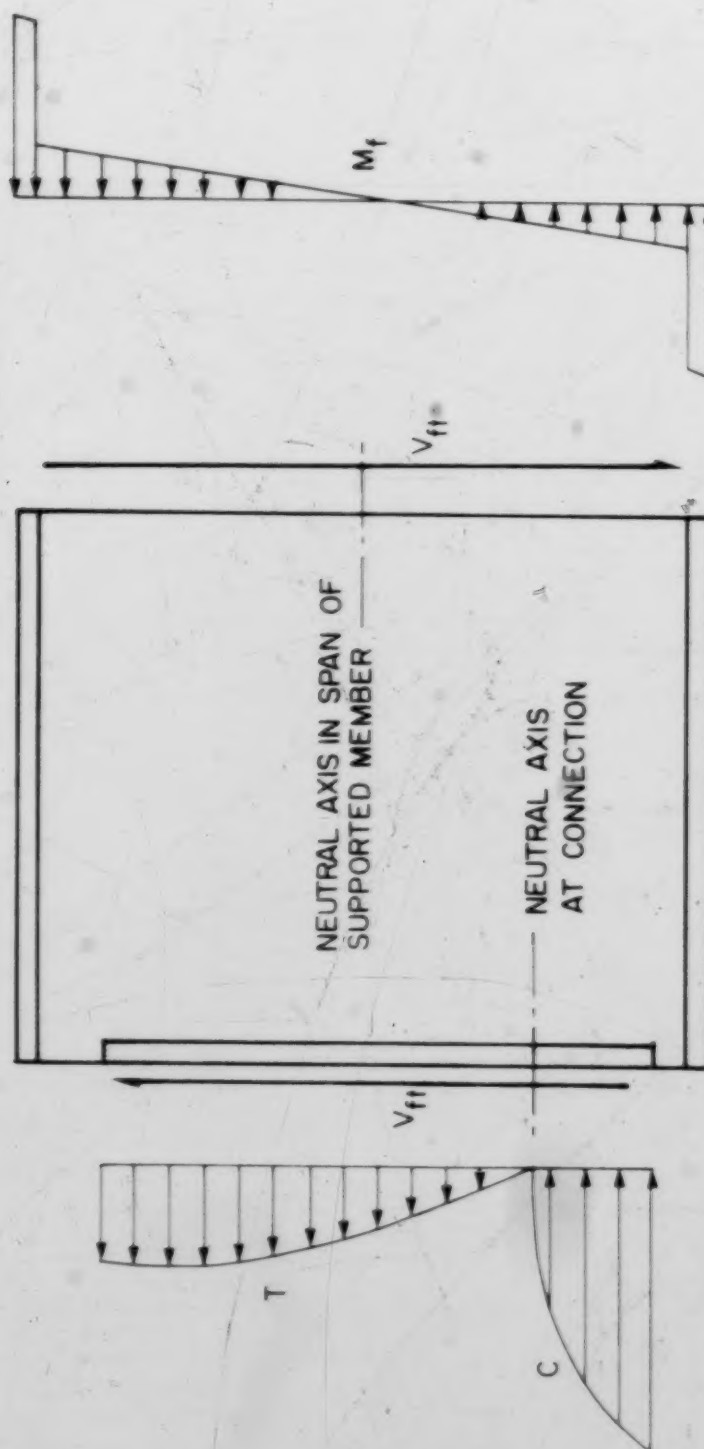


Figure 6.3 Force Distributions at the Connection and Within the Span of a Member Supported by a Flexible End Plate Connection

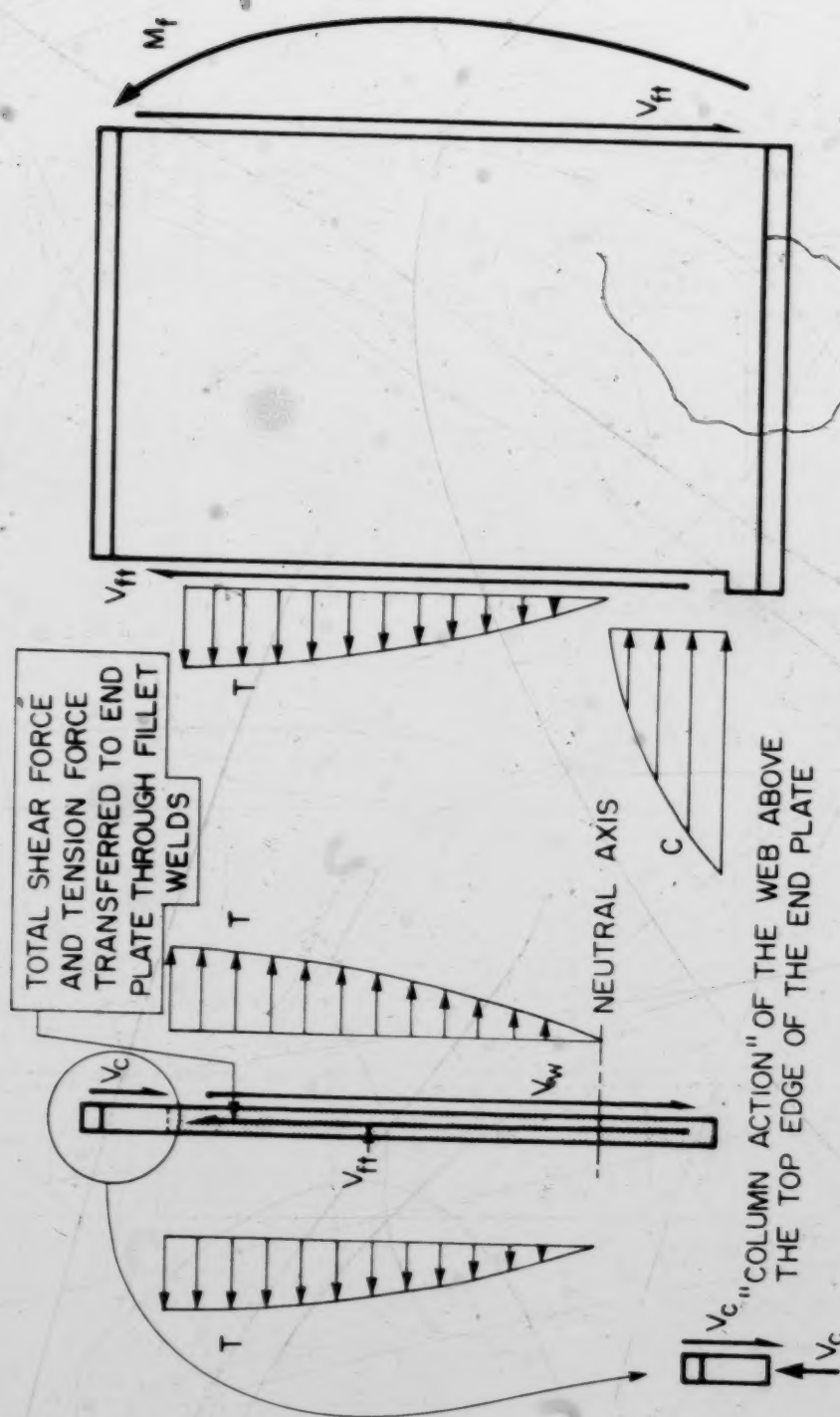


Figure 6.4 Shear Transfer Path From the Web of an Ungroped Member to the Fillet Welds Joining the Supported Member to a Flexible End Plate Connection

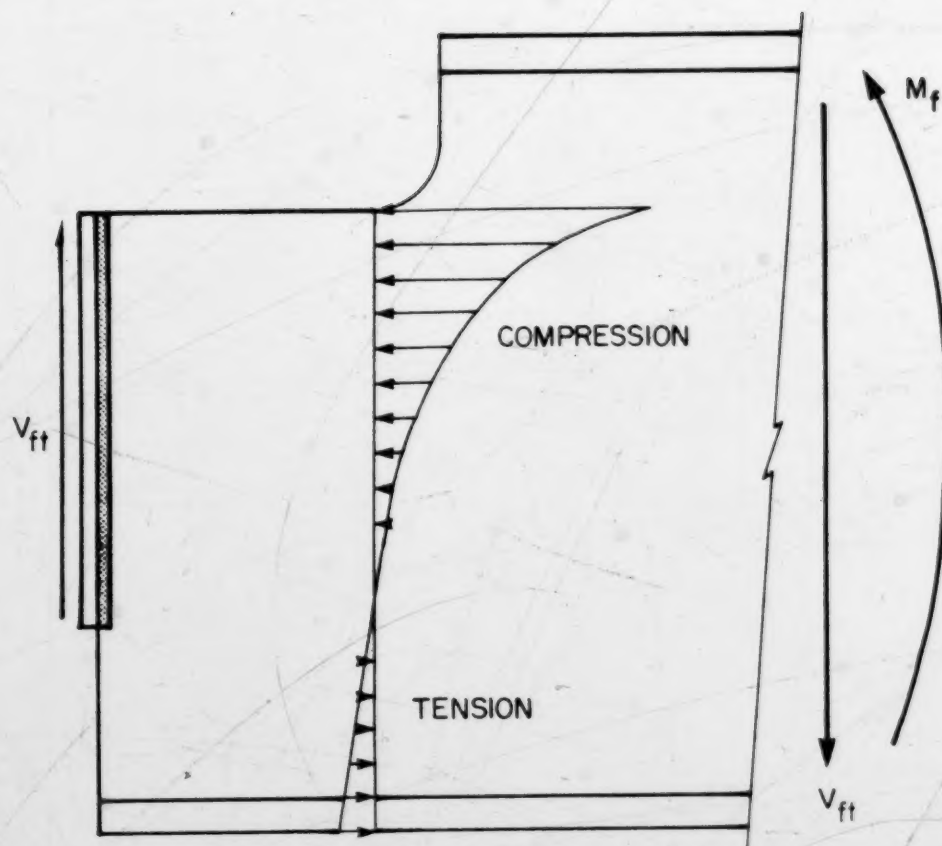


Figure 6.5 Possible Flexural Stress Distribution at Coped Region of a Simply Supported W-Shape

PART B: DEVELOPMENT OF STANDARDIZED FLEXIBLE END PLATE CONNECTIONS

7. Requirements for Standardized Connections

7.1 Introduction

Standardized connections can be described as a collection of pre-designed connections with prescribed geometric details and material characteristics and with specified strengths or capacities. Although industry-wide standardized connections currently available meet these criteria, the use of the high-speed computer with an adequate model of the connection behaviour allows a much broader range of details to be considered and indeed,

1. connections can be standardized to meet the shop practice of a particular fabricator,
2. connection costs can be developed, again based on individual shop costs, and
3. more accurate design models can be used.

7.2 Limitations of Current Approach to Standardized Connections

Designers use handbooks such as the "Handbook of Steel Construction" (CISC 1980) to select steel members once the factored forces acting on the members, have been determined. The handbooks give the factored resistances of

the members based on the relevant design standard. When it comes to selecting connection details, designers again can choose appropriate connections from a list of pre-designed connections. These connection design tables are not likely to reflect the fabrication techniques preferred by a particular fabricator and are therefore restrictive and can result in increased costs.

A multitude of combinations of plates, bolts, welds, gages, and pitches exist, even for one type of connection. Up to the present time, only a limited number of combinations of connection components for flexible end plate connections have been proposed. Some of these are shown in Table 7.1 (AustISC 1978, BCSA 1982, SAISC 1982).

In North America both the Canadian Institute of Steel Construction (CISC) and the American Institute of Steel Construction (AISC) provide a slightly different approach. Instead of publishing tables of pre-designed connections, they provide tables of connection component strengths that can be used as 'building blocks' to develop shop standards. Even though these tables are based on a limited number of possible component geometries and material strengths, a large number of different connection details can be developed. However, it is expensive, even with these data, for fabricators to develop standardized connections.

A preferred alternative to using published standard connection details is for fabricators to be able to develop their own sets of pre-designed details based on the use of

the high-speed computer with rational connection design models. Also, by using computers the variation in fabrication costs can be considered when optimizing connection details. The advent of Limit States Design, which recognizes serviceability limit states and ultimate limit states, provides additional impetus to develop models that can describe connection behaviour throughout the loading range. Again, this is facilitated by high-speed computational devices.

A review of the connection selection process using current pre-designed connection details identifies other limitations and the considerable work involved. The steps the designer takes are as follows:

1. Determine the type of supported member, its end preparations such as copes, end shear force, and the type of supporting member.
2. From tables choose connection details of sufficient capacity.
3. Make a series of supplementary connection design checks, including the evaluation of the bearing capacity of the supporting element at the bolts, the reduced shear and flexural capacity due to coping, and the rotational serviceability of the connection. These may be aided by other design tables.
4. Ensure that the chosen connection will fit the support conditions.

By developing standardized connections based on consideration of beam size, span length, and loading, the limiting rotation of the connection can be evaluated directly. This eliminates the need for a supplementary check, as is currently made in the Australian connection design handbook (AustISC 1978). The other supplementary design checks can also be included in a computer method, and can thereby reduce not only the design time, but also the possibility of errors.

Thus, it would seem most promising to produce computer software that could be used to generate standardized connection details of the form discussed here. Fabricators would have standardized connections meeting their specific preferences. To date this approach has not been implemented.

7.3 Proposed Interactive Computer Program for Standardized Connections

Computer software can be developed to provide 'customized' connection design tables, one possible example of which is given in Appendix B for flexible end plates. The program is written in Fortran IV language. In addition it has certain system commands that are compatible with the Michigan Terminal System (MTS) operating system as used at the University of Alberta. Two different connection design models have been included:

1. the proposed Limit States Design (LSD) model as developed in PART A, and
2. the simple vertical shear (SVS) model as used by the CISC in preparation of its flexible end plate component strength design tables (CISC 1980).

Input data can be introduced to the program either in the form of a data file or during an interactive input session. The program uses metric units exclusively, and is based on the CSA Standard CAN3-S16.1-M78, "Steel Structures For Buildings Limit States Design" (CSA 1978).

The program is linked with the data files prepared by the Canadian Institute of Steel Construction (CISC) of standard rolled and welded shapes. Therefore only a minimum of data concerning the geometry of the supported and supporting members needs to be input by the user. Connections can be designed for WWF, W, HP, M, S, C, and MC shapes as listed in the "Handbook of Steel Construction" (CISC 1980). Material properties of the supported and supporting members can be selected by the program user.

The program computes the factored end shear forces for the specified span lengths and loading conditions. For the LSD model of PART A the end rotation is also required and it is therefore expedient to input the information from which both of these quantities can be calculated. The user specifies the load type as being uniformly distributed or as

being single concentrated (load type combinations are not permitted) and the load level up to 100 percent of the ultimate load. With this information, connections for ten different span lengths of the supported member, ranging from three times the beam depth to thirty times the beam depth in increments of three times the beam depth, are developed.

For a given beam span and load type, the design end shear force for the connection is determined using the smaller of either the flexural or shear strength of the supported member at the load level specified. All calculations are based on the assumptions that continuous lateral support is present and the end conditions of the supported member are perfect hinges, one of which has no translational stiffness.

Geometric and material properties for the connection plate, bolts, and fillet welds are selected by the user, with the exception of the connection plate length, which is determined, by the program, based on the other parameters. Cope geometry can be specified by the user. The user can assign cost factors to the three basic connection components in order to determine total connection costs.

The program output is extensive and includes detailed information on the geometric and material properties of both the supported member and the connection together with a series of tabulated connection analysis and design information.

Table 7.1 Geometric and Material Property Limitations on
Standardized Flexible End Plate Connections

	CHOICES AVAILABLE		
	AustISC (1978)	BCSA (1982)	SAISC (1982)
END PLATE:			
thicknesses	1	2 ^a	2
material grades	1	1	1
gages	1	not set	2 ^c
itches	1	1	2 ^c
end distances	1	1	1
edge distances	1	2 ^b	1
BOLTS:			
diameters	1	1	1
material grades	2	2	2
WELDS:			
sizes	1	2 ^c	3 ^d
material grades	1	1	1
SUPPORTED BEAM:			
material grades	1	2	1

- Depends on:
- a size of the supported member,
 - b plate cutting process,
 - c bolt grade used,
 - d supported member web thickness.

8. Interactive Computer Program: Internal Operations

8.1 Introduction

The interactive computer program for the *Connection Design of Simply Supported Steel Shapes* (CDSSSS), given in Appendix B, designs flexible end plate connections using either the Limit States Design (LSD) model of PART A or the Simple Vertical Shear (SVS) model from the "Handbook of Steel Construction" (CISC 1980). The model from PART A cannot be used effectively without the use of a high-speed computational device because numerical iterative procedures are essential in the design process. Figure 8.1 shows a general flow chart of the internal operations used in the CDSSSS program.

8.2 Input Data

The CDSSSS program first reads into data storage locations the necessary input parameters from

1. a separate data file that must be referenced when program execution is initiated, or
2. a computer terminal during execution of CDSSSS when the interactive data input mode is used.

The second input method is preferred because an explanatory 'user manual' is not needed. The interactive

mode has sufficient explanations and 'error traps' built into it to notify the user when input data is unacceptable.

The necessary input data requires:

A. For the members,

1. identification of the supported member(s) in one of three options:
 - i. one standard shape, eg. a W360X33 shape,
 - ii. one subcategory of standard shapes, eg. all W360 shapes,
 - iii. one category of shapes, eg. all W shapes,
2. identification of the supporting element, in one of four options:
 - i. flange of a column,
 - ii. web of a column,
 - iii. web of a beam,
 - iv. a steel plate element of a given thickness,
3. specified yield strength of the supported and supporting members (MPa),
4. specified modulus of elasticity of the supported member (MPa),
5. resistance factor of the supported member,
6. top and bottom flange cope dimensions of the supported member, if any (mm),
7. type of loading, it being either a uniformly distributed or a concentrated load, and the percent

of full factored loading,

B. For the connection,

8. connection model type, that is either:
 - i. Simple Vertical Shear (SVS) model,
 - ii. Limit States Design (LSD) model,
9. specified bolt strength (MPa),
10. specified bolt diameter (mm),
11. ϕ , resistance factor for bolts,
12. bolt cost factor (\$/bolt),
13. specified electrode strength (MPa),
14. specified fillet weld leg size (mm),
15. ϕ , resistance factor for welds,
16. weld cost factor (\$/mm of weld),
17. specified yield strength of the connection plate (MPa),
18. specified ultimate strength of the plate (MPa),
19. specified modulus of elasticity of the plate (MPa),
20. ϕ , resistance factor for plate,
21. plate cost factor (\$/kg),
22. bolt hole gage (mm),
23. bolt hole pitch (mm),
24. bolt hole end distance (mm),
25. bolt hole edge distance (mm),
26. bolt hole diameter (mm),
27. distance from the top row of bolts to the top of the

- supported member (mm),
- 28. end plate thicknesses (mm)(from one to four can be specified),
- 29. a table numbering identification number for the output.

When the interactive input feature of the program is used, reuse of any or all of the input data from the previous program operation which is stored within a data file attached to CDSSSS reduces the amount of data that must be input for each new program use. The program also requires a complete list of the geometric properties of the supported member or members. This is obtained automatically from a data file containing all of the geometric properties of all of the standard steel shapes as given in the "Handbook of Steel Construction" (CISC 1980).

8.3 Classification and Factored Resistance of Supported Member

From the type of load and the percent of full factored load that the supported member carries, the end shear force is determined. This is based on the shear and on the flexural resistance of the supported member as found from the resistance equations in CSA Standard CAN3-S16.1-M78 (CSA 1978). The section class is, of course, taken into account and the class of a member is included in the

connection design tables for reference. It is assumed that all beams are laterally supported.

8.4 Determination of Loading Magnitude and Service Level Deformations of Supported Member

Rather than have the program user specify a span length for the supported member, the CDSSSS program designs connections for a series of ten different span lengths of the supported member. These are established in ten equal increments over the range of three times the depth to thirty times the depth of the supported member.

For each of the ten span lengths, with the load type, the percentage of ultimate loading, and the member strength, it is possible to calculate the corresponding magnitude of the factored load level. Using this factored load and the dead load factor of 1.25 (CAN3-S16.1-M78 (CSA 1978)), the magnitude of the unfactored or specified 100 percent dead load is calculated.

Beam end rotations and maximum span deflections at the 100 percent specified dead load level are calculated from moment-area analyses (see, for example, Popov (1978)) assuming the beam to be simply supported and to be carrying either uniformly distributed or single concentrated loads. This information, as is discussed later, is considered useful for the development of other design information and is therefore included in the CDSSSS program.

8.5 Determination of Supported Member Deformations at the Factored Load Level

When the Limit States Design (LSD) model proposed in PART A is specified by the program user, the connection end rotation at the factored load level must be calculated as the design of the connection depends upon this rotation. Although this information is not used in the evaluation of connection strength when the SVS model is specified, it is calculated for this case also. The maximum span deflection is also calculated under the factored load.

When the factored load does not cause flexural yielding the end rotations and span deflections are determined using moment-area principles (see, for example, Popov 1978). For simplicity, the reduced deflections due to the end moments developed are neglected. As well, the effect of shear is ignored.

If the factored load causes flexural yielding to occur, then the moment-area method no longer applies for those portions of the beam where some yielding has occurred. For these portions it is assumed that the stress-strain relationship is elasto-plastic and residual stress effects do not exist. The curvature at a distance, 'x' along the beam is given by:

$$[8.1] \quad \phi(x) = \frac{2 \epsilon_{\max}(x)}{d}$$

which can be rearranged (see Figure 8.2) to:

$$[8.2] \quad \Phi(x) = \frac{2}{(d - 2e(x))} \frac{\phi \sigma_y}{E}$$

Calculation of $\Phi(x)$ using Equation [8.2] requires a value of $e(x)$. Since it is inconvenient to calculate $e(x)$ given $M(x)$, the method used in the CDSSS program to determine $e(x)$ is an iterative one. Knowing $M(x)$, a series of different $e(x)$ values are assumed until the internal bending moment $M_1(x)$ matches the external bending moment $M(x)$. For W and C shapes (see Figure 8.3) the inelastic moments, when yielding is confined to the flanges and when it extends into the web, are respectively:

$$[8.3] \quad M_1(x) = \phi \sigma_y (d - e(x)) b e(x) + \left(\frac{2 \phi \sigma_y}{d - 2e(x)} \right) \left\{ \frac{(d - 2t)^3}{12} w + \frac{b}{2} (t - e(x)) (d - t - e(x))^2 \right\}$$

and:

$$[8.4] \quad M_1(x) = \phi \sigma_y (d - t) b t + \phi (e(x) - t)(d - e(x) - t) w \sigma_y + \phi (d - 2e(x))^2 \frac{w \sigma_y}{6}$$

In recognition of strain-hardening, $\epsilon_{max}(x)$ has been taken as $10\epsilon_y$ when M_p is reached. This level of strain corresponds to that attainable by class 1 and 2 shapes when the onset of local buckling is imminent ("Plastic Design in Steel", ASCE 1971).

The curvatures, taking note of the boundary conditions, are summed to determine slopes. The slopes are then integrated to determine the maximum connection rotations

and span deflection.

8.6 Design of Flexible End Plate Connections

The CDSSSS program first selects connection details assuming that only uniformly distributed vertical shear acts on the connection. The connection size is governed by the bolts in shear or bearing, the end plate in shear, the fillet welds in longitudinal shear, or the supported member web in shear only. For the SVS model, which does not consider the effect of connection deformations on the strength, this step completes the design process. For the LSD model, with this preliminary connection size determined, the remaining connection design checks are made. This includes an evaluation of the limiting rotation of the connection and a consideration of the effect that the flexural forces developed at the connection have on its vertical shear capacity. The iterative procedure shown in the flow chart of Figure 2.4 is followed until a satisfactory connection detail is obtained.

8.7 Program Output: Standardized Connection Design Tables

The output generated for the standardized connection details for one supported member type is extensive, as shown in Table 8.1. The first page of output contains input data summaries and some general design information regarding the supported member. The second and third pages contain the standardized connection details.

The top of the first page provides a complete description of the supported member, including geometric and material properties of the section as well as the class and the strength of the section. Width-to-thickness classifications for the flange and for the web of the supported member are given. For the web, these alpha-numeric classifications contain a 'B' when referring to flexure and contain a 'V' when referring to shear.

The second block of information shown on the first page of Table 8.1 is a complete description of the geometric and material properties, the unit resistances, and the cost factors for each of the connection components: the plate, the bolts, and the welds. At the top of this connection data summary, the expression

FEP(Fyp-Dh-g-p-Dend-Dedg-Dtop-tp-L) B(Fub-Db-#) W(Xu-f)

completely describes the flexible end plate connection once the connection is designed. On page 2 of Table 8.1 these symbols are replaced, generally, by their numeric values with the exceptions of the plate thickness (tp), the plate length (L), and the number of bolts in a connection (#), which vary for different connection details and are therefore not given at this point in the design table.

Based on the assumption that the supported member is simply supported, information is given at the bottom of the first page which pertains to the loading and deformation of

the supported member at service and ultimate levels.

The information is as follows:

1. supported member length,
2. supported member length-to-depth ratio,
3. factored and 100 percent service dead load,
4. ratio of the supported member shear resistance to the applied factored shear force,
5. ratio of the supported member moment resistance to the maximum applied factored moment,
6. maximum span deflection of the supported member at factored and at 100 percent service dead load,
7. ratios of the supported member span to the maximum span deflection at factored and at 100 percent service dead load, and
8. the supported member end rotation at factored and at 100 percent service dead load.

The remaining pages of the output contain the actual connection details. Up to four connection plate thicknesses can be considered separately by the CDSSSS program. For each end plate thickness connection details are tabulated for each of the ten span lengths. Except for the plate length (L) and the number of bolts (#) all geometric and material properties are identical for all ten span lengths. These properties are summarized at the heading of each design table in the short form expression described earlier. In addition, the design model type, ie. LSD or SVS, the support

type, and the cope dimensions are all listed at the heading of each table of connection details.

For each of the ten beam span lengths the following connection detail information is provided in tabular form:

1. supported member length,
2. supported member length-to-depth ratio,
3. required end plate length,
4. required number of bolts,
5. connection cost,
6. condition which governed the connection size,
7. maximum factored end shear force,
8. ratios of the connection component resistances to the factored shear force,
9. factored level deformation at the top of the end plate due to connection rotation,
10. 100 percent service dead load level deformation at the top of the supported beam due to connection rotation,
11. end moment at the factored rotation associated with the recommended connection detail,
12. end moment at the factored rotation associated with the maximum length connection detail (this may or may not be the same as the recommended detail),
13. ratios of the two end moments to the moment resistance of the supported member, and
14. adjusted factored level load that the supported member can carry when the end moments are considered.

When the SVS model is used certain values in the output table are not printed because they do not apply.

Table 8.1 Standardized Connection Design Table for the
Flexible End Plate Connection

(a) Output Page 1

W8x22 (Available in Canada And U.S.A. is Normally Used As A Beam)	
SECTION PROPERTIES	
Sectional Area	A = 21 Kg/m
Gross Sectional Area	A _g = 7750 mm ²
x-Axis Moment of Inertia	I _x = 258 0x10 ⁶ mm ⁴
x-Axis Section Modulus	S _x = 1180 0x10 ³ mm ³
x-Axis Plastic Modulus	I _{px} = 1310 0x10 ³ mm ³
y-Axis Moment of Inertia	I _y = 12 2x10 ⁶ mm ⁴
y-Axis Section Modulus	S _y = 128 0x10 ³ mm ³
y-Axis Plastic Modulus	I _{py} = 302 0x10 ³ mm ³
Torsional Constant	J = 288 0x10 ³ mm ⁴
Warping Constant	C _w = 588 0x10 ⁶ mm ⁶
Overall Section Depth	d = 450.0 mm
Flange Width	b _f = 159.0 mm
Flange Thickness	t _f = 10.8 mm
Clear Flange Distance	FT = 75.5 mm
Web Height	h _w = 429.4 mm
Web Thickness	w _t = 8.1 mm
Clear Web Distance	WT = 394.0 mm
MATERIAL PROPERTIES	
Yield Strength	F _y = 300.0 MPa
Modulus of Elasticity	E = 200000 MPa
SECTION CLASS & SECTION STRENGTH	
Flange Width/Thickness, b/t	= 8.7, CLASS 2
Web Width/Thickness, h/w	= 52.8, CLASS 1B
Resistance Factor	φ = 0.90
Fact. Vid. Moment Cap.	M _v = 310.5 kNm
Fact. Moment Capacity	M _r = 383.7 kNm
Fact. Shear Capacity	V _r = 885.5 kN
Max. Shear Strength	P _s = 198.0 MPa
	= 0.65F _y

FLEXIBLE END PLATE CONNECTION		FEP(Fyp-Dh-g-Dend-Gedg-Stop-tp-L) S(Fub-Dh-w) W(Xu-F)	
PLATE PROPERTIES		BOLT PROPERTIES	
Yield Strength	Fyp = 300.0 MPa	Ultimate Strength	Fub = 535.0 MPa
Ultimate Strength	Fup = 450.0 MPa	Resistance Factor	φ _b = 0.87
Modulus of Elasticity	E _p = 200000.0 MPa	Spinal Diameter	DB = 19.0 mm
Resistance Factor	φ _p = 0.90	Fact. Tensile Cap.	T _{DB} = 117.5 kN/Bolt
Bolt Hole Diameter	Dh = 21.0 mm	Fact. Shear Capacity	V _{DB} = 85.8 kN/Bolt
Bolt Hole Gage	G = 125.0 mm	Bolt Cost	COSTB = \$ 10.00/Bolt
Bolt Hole Pitch	P = 70.0 mm	WELD PROPERTIES	
Bolt Hole End Dist.	Dend = 35.0 mm	Ultimate Strength	Xu = 450.0 MPa
Bolt Hole Edge Dist.	Gedg = 30.0 mm	Resistance Factor	φ _w = 0.87
Bolt Hole Top Dist.	Stop = 75.0 mm	Pillet Weld Leg Size	F = 8.0 mm
Material Cost	COSTP = \$ 1.00/kg	Fact. Long. Sh. Cap.	V _{FW} = 0.91 kN/mm
		Pillet Weld Cost	COSTW = \$ 0.10/mm

UNIFORMLY DISTRIBUTED LOAD @ 55 % OF THE ULTIMATE LOAD LEVEL FOR THIS MEMBER.												
#	Beam Length (mm)	Length Depth	FACT LOAD (kN/m)	SPEC 100%DL (kN/m)	V _r V _f	M _r M _f	Max FL Defl (mm)	Max DL Defl (mm)	Length FLOEFF	Length DLOEFF	End FL (kN)	End DL (kN)
1	350	3.0	914.2	731.3	1.1	1.7	0.8	0.8	1789	2211	0.0018	0.0014
2	2700	8.0	368.7	295.0	1.3	1.1	6.8	3.9	293	888	0.0074	0.0067
3	4050	9.0	163.8	131.1	2.0	1.1	14.8	8.9	281	457	0.0108	0.0070
4	5400	12.0	92.2	73.7	2.8	1.1	27.5	15.8	187	383	0.0148	0.0093
5	6750	15.0	58.0	47.2	3.3	1.1	41.3	24.8	184	278	0.0182	0.0117
6	8100	18.0	41.0	32.8	3.9	1.1	61.4	35.5	131	228	0.0232	0.0140
7	9450	21.0	30.1	24.7	4.5	1.1	81.7	48.3	118	198	0.0258	0.0163
8	10800	24.0	23.0	18.4	5.2	1.1	109.9	63.1	88	171	0.0287	0.0187
9	12150	27.0	18.2	14.8	5.9	1.1	138.2	79.8	89	182	0.0330	0.0210
10	13500	30.0	14.7	11.8	6.5	1.1	171.7	98.5	78	137	0.0371	0.0234

Table 8.1 Standardized Connection Design Table for the Flexible End Plate Connection

(b) Output Page 2

W360x75 (Available in Canada And U.S.A.), Normally Used As A Beam											
CONN #1 : FEP(300-21-125- 70-35-30- 75- 8-L) S: 825-19-W: W(480- 8) L50											
Support: Flange of a W360x75 Column, 15.8 mm thick. Material Prop. as per Supported Member. Supported member is uncoped											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V7	Vweld V7	Valt V7	Vbeam V7
1	1350	3.0	350.0	10	173.04	Beam	817.1	1.03	1.10	0.99	0.83
2	2700	5.0	350.0	10	173.04	Beam	497.8	1.37	1.39	1.23	1.00
3	4050	8.0	350.0	8	138.43	Beam	331.9	1.93	1.99	1.49	1.31
4	5400	12.0	310.0	8	103.43	Beam	248.9	1.93	1.70	1.49	1.24
5	6750	15.0	280.0	8	138.43	Rate	199.1	3.84	3.77	3.49	2.01
6	8100	18.0	280.0	8	138.43	Rate	168.9	3.09	3.33	2.95	2.40
7	9450	21.0	350.0	10	173.04	Rate	142.2	4.45	4.87	4.31	3.41
8	10800	24.0	350.0	10	173.04	Rate	124.4	5.09	5.98	4.92	3.88
9	12150	27.0	350.0	10	173.04	Rate	110.8	5.72	6.26	5.54	4.19
10	13500	30.0	350.0	10	173.04	Rate	99.5	6.45	6.50	5.20	4.18
#	Beam Top Flt. FL (mm)	Beam Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 Mr	Mend2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	0.4	0.4	15.7	15.7	0.04	0.04	982.3	982.3			
2	2.1	1.9	22.4	22.4	0.08	0.08	393.3	393.3			
3	3.5	1.8	19.1	23.6	0.04	0.07	171.3	175.4			
4	2.7	2.0	8.8	24.8	0.02	0.07	94.8	95.0			
5	4.4	3.2	16.2	28.2	0.05	0.07	81.8	83.6			
6	5.4	3.9	17.2	27.9	0.05	0.08	43.1	44.4			
7	7.7	5.5	29.8	29.8	0.08	0.08	32.8	32.8			
8	8.9	6.2	31.7	31.7	0.08	0.08	26.2	26.2			
9	9.7	7.0	33.8	33.8	0.09	0.09	20.0	20.0			
10	Flng	Flng	Flng	Flng	Flng	Flng	Flng	Flng			

CONN #2 : FEP(300-21-125- 70-35-30- 75- 8-L) S: 825-19-W: W(480- 8) L50											
Support: Flange of a W360x75 Column, 15.8 mm thick. Material Prop. as per Supported Member. Supported member is uncoped											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V7	Vweld V7	Valt V7	Vbeam V7
1	1350	3.0	350.0	10	174.06	Beam	817.1	1.07	1.08	1.32	0.81
2	2700	5.0	350.0	10	174.06	Beam	497.8	1.32	1.32	1.84	0.98
3	4050	8.0	280.0	8	139.24	Beam	331.9	1.99	1.99	1.97	1.18
4	5400	12.0	210.0	8	104.43	Beam	248.9	1.99	1.98	1.97	1.18
5	6750	15.0	210.0	8	104.43	Rate	199.1	1.98	1.98	2.48	1.48
6	8100	18.0	280.0	8	139.24	Rate	168.9	3.17	3.17	3.94	2.23
7	9450	21.0	280.0	8	139.24	Rate	142.2	3.70	3.69	4.59	2.80
8	10800	24.0	350.0	10	174.06	Rate	124.4	5.39	5.25	6.58	3.44
9	12150	27.0	350.0	10	174.06	Rate	110.8	5.92	5.86	7.38	3.82
10	13500	30.0	350.0	10	174.06	Rate	99.5	6.45	6.50	8.20	4.18
#	Beam Top Flt. FL (mm)	Beam Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 Mr	Mend2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	0.4	0.4	25.3	25.3	0.07	0.07	982.3	982.3			
2	1.9	1.3	38.3	38.3	0.10	0.10	407.4	407.4			
3	3.2	1.7	23.7	38.8	0.07	0.10	178.8	181.8			
4	2.4	1.8	14.1	38.4	0.04	0.11	95.1	102.7			
5	3.0	2.3	14.4	38.4	0.04	0.11	81.5	85.9			
6	4.9	3.5	28.1	41.2	0.07	0.13	44.2	45.0			
7	5.7	4.1	28.8	42.7	0.08	0.12	32.9	33.9			
8	8.1	5.4	44.7	44.7	0.13	0.13	26.1	26.1			
9	9.0	5.9	48.4	48.4	0.13	0.13	20.7	20.7			
10	10.1	7.3	48.8	48.4	0.14	0.14	18.9	18.9			

Table 8.1 Standardized Connection Design Table for the
Flexible End Plate Connection

(c) Output Page 3

W360X51 (Available in Canada And U.S.A.), Normally Used As A Beam											
CONN #3 : FEP(300-21-125- 70-35-30- 75-10-L) B(825-19-W) W(480- 8) LSD											
Support, Flange of a W360X79 Column, 15.8 mm thick. Material Prop. as per Supported Member											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vplt V _f	Vbeam V _f
1	1350	3.0	350.0	10	175.07	Beam	817.1	0.98	0.88	1.85	0.78
2	2700	8.0	350.0	10	175.07	Beam	427.4	1.21	1.10	2.05	0.88
3	4050	9.0	350.0	8	140.06	Beam	331.8	1.42	1.27	2.48	1.07
4	5400	12.0	210.0	8	105.04	Beam	248.8	1.44	1.30	2.48	1.09
5	6750	15.0	210.0	8	105.04	Beam	199.1	1.80	1.82	3.08	1.34
6	8100	18.0	210.0	8	105.04	Beam	185.8	2.18	1.94	3.88	1.69
7	9450	21.0	280.0	8	140.06	Beam	142.2	3.30	2.94	5.74	2.28
8	10800	24.0	280.0	8	140.06	Beam	124.4	3.78	3.38	6.88	2.52
9	12150	27.0	350.0	10	175.07	Beam	110.8	5.08	4.57	9.23	3.38
10	13500	30.0	350.0	10	175.07	Beam	99.8	5.91	5.03	10.25	3.88
#	Dend Top Plt. FL (mm)	Dend Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 M _r	Mend2 M _r	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	0.3	0.3	35.2	35.2	0.10	0.10	982.3	982.3			
2	1.7	1.2	48.9	48.9	0.14	0.14	422.4	422.4			
3	2.0	1.5	33.0	31.3	0.09	0.15	180.0	188.8			
4	2.1	1.6	18.8	53.3	0.06	0.15	97.8	108.8			
5	2.7	2.1	20.0	54.4	0.06	0.15	82.5	88.8			
6	3.4	2.8	20.4	55.3	0.06	0.15	43.5	47.8			
7	3.2	3.9	35.5	57.2	0.10	0.18	32.4	35.2			
8	5.0	4.4	37.6	59.1	0.11	0.17	35.6	37.1			
9	6.3	6.0	60.8	60.8	0.17	0.17	21.5	21.5			
10	9.4	6.6	62.4	62.4	0.18	0.18	17.5	17.5			

CONN #4 : FEP(300-21-125- 70-35-30- 75-12-L) B(825-19-W) W(480- 8) LSD											
Support, Flange of a W360X79 Column, 15.8 mm thick. Material Prop. as per Supported Member											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vplt V _f	Vbeam V _f
1	1350	3.0	350.0	10	175.08	Weld	817.1	0.80	0.88	1.85	0.78
2	2700	8.0	350.0	10	175.08	Weld	427.4	0.83	0.82	2.48	0.78
3	4050	9.0	350.0	10	175.08	Weld	331.8	1.24	0.82	3.89	1.10
4	5400	12.0	280.0	8	140.87	Weld	248.8	1.48	1.17	3.94	1.14
5	6750	15.0	280.0	8	140.87	Beam	199.1	1.82	1.48	4.92	1.39
6	8100	18.0	280.0	8	140.87	Weld	185.8	1.87	1.08	5.90	1.83
7	9450	21.0	210.0	8	105.85	Beam	142.2	1.81	1.14	5.17	1.50
8	10800	24.0	280.0	8	140.87	Beam	124.4	2.17	1.42	7.87	2.08
9	12150	27.0	350.0	10	175.08	Beam	110.8	3.41	1.88	8.88	2.31
10	13500	30.0	350.0	10	175.08	Beam	99.8	3.39	2.90	12.20	2.88
#	Dend Top Plt. FL (mm)	Dend Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 M _r	Mend2 M _r	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	0.3	0.3	44.7	44.7	0.13	0.13	982.3	982.3			
2	1.5	1.0	82.8	82.8	0.18	0.18	437.4	437.4			
3	2.3	1.7	85.2	85.2	0.18	0.18	195.7	195.7			
4	2.8	1.8	43.8	87.8	0.13	0.18	104.1	110.8			
5	3.2	2.4	48.0	70.1	0.13	0.20	88.8	71.3			
6	4.0	2.9	48.9	71.8	0.13	0.20	48.8	48.7			
7	3.4	3.8	35.9	73.2	0.08	0.21	32.5	38.7			
8	5.5	4.0	47.8	74.3	0.13	0.21	28.3	28.1			
9	6.1	4.6	48.5	75.8	0.14	0.21	20.8	22.3			
10	8.8	6.2	77.8	77.8	0.22	0.22	18.2	18.2			

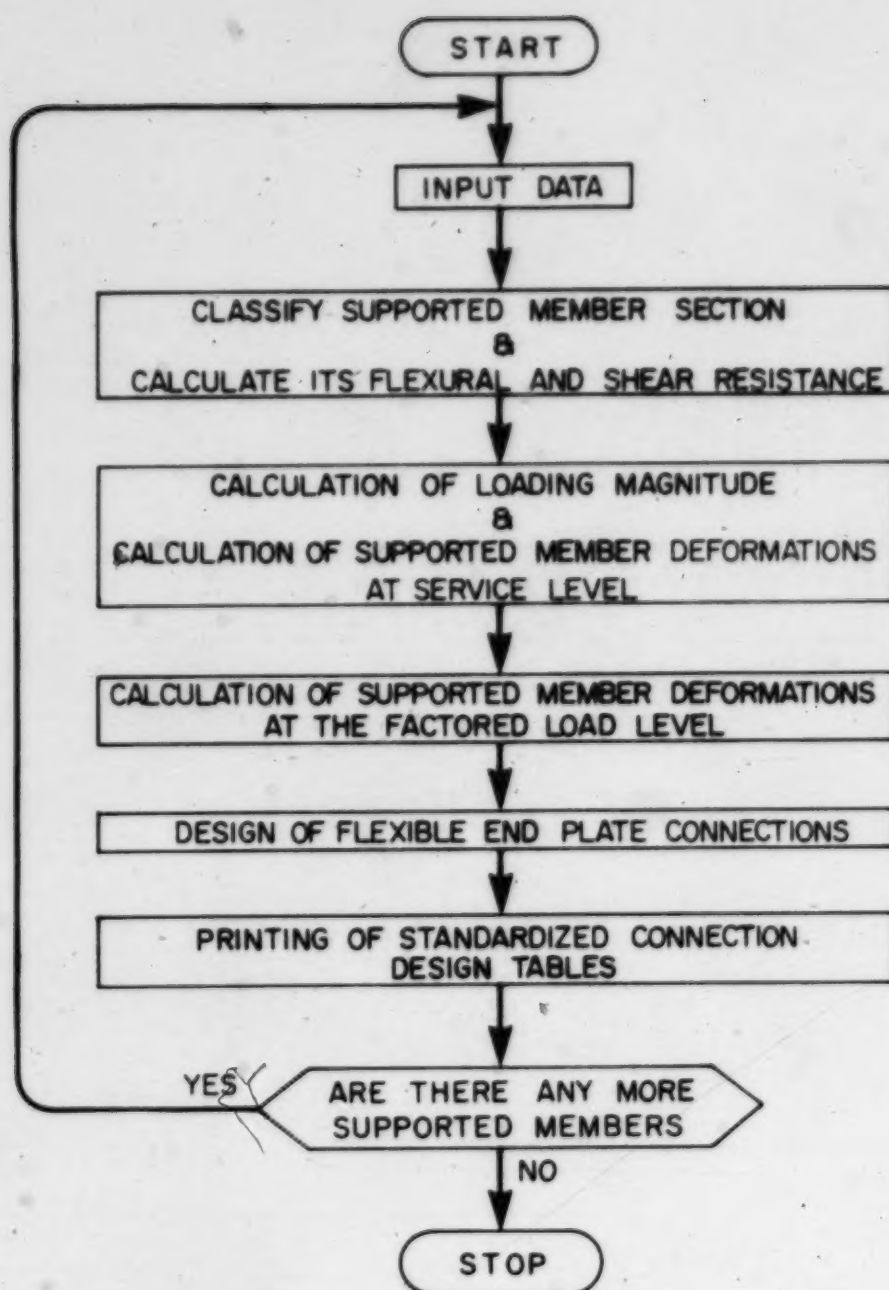
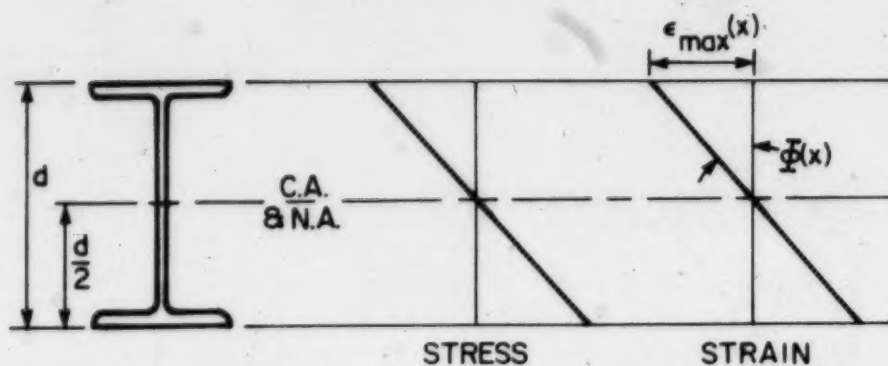
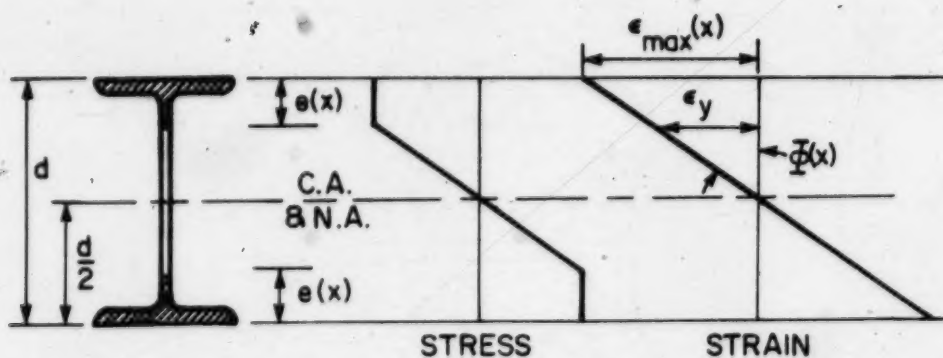


Figure 8.1 Flow Chart of the Interactive Computer Program
for the Design of Standard Flexible End Plate
Connection Details

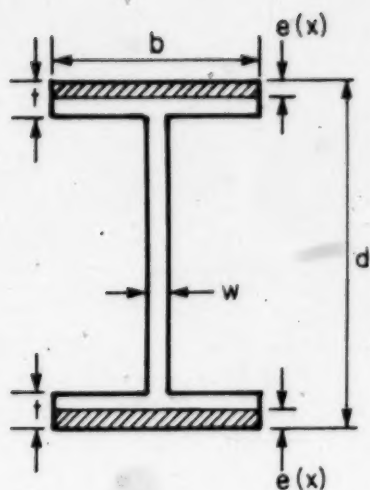


a) ELASTIC LEVEL, $\epsilon_{\max} < \epsilon_y$



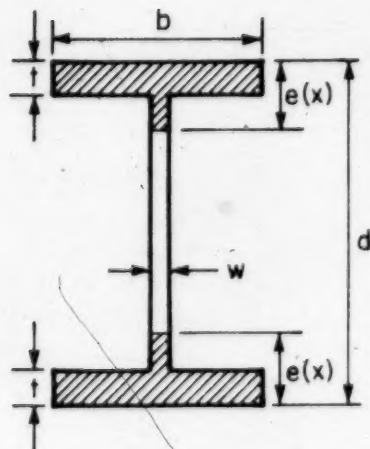
b) INELASTIC LEVEL, $\epsilon_{\max} > \epsilon_y$

Figure 8.2 Determination of Curvature at Location x of a Cross-Section



$$M_i(x) = \phi(d-e(x)) b e(x) \sigma_y + \frac{2 \phi \sigma_y}{(d-2e(x))} \left\{ (d-2t)^3 \frac{w}{12} + (t-e(x)) \frac{b}{2} (d-t-e(x))^2 \right\}$$

i) YIELDING CONFINED TO FLANGES



$$M_i(x) = \phi(d-t) b t \sigma_y + \phi(e(x)-t)(d-e(x)-t) w \sigma_y + \phi(d-2e(x))^2 \frac{w \sigma_y}{6}$$

ii) YIELDING INTO THE WEB

Figure 8.3 Calculation of the Internal Moment at a Cross-Section Which is Partly Inelastic

9. Interactive Computer Program: Design Table Use and Applications

9.1 Introduction

Standardized connection design tables developed using the CDSSSS program can be used by both fabricators and consulting engineers, either for a given project or for general usage. Four example connection design tables are given in Tables 9.1 to 9.4. These tables, each of which is three pages long, were developed using the LSD model for uncoped W530x82 members supporting uniformly distributed loads. The loads range from 85 to 100 percent of ultimate levels in increments of 5 percent. Thus, each of the tables provides connection designs for the supported member at a factored load level causing a different percentage of the factored resistance of that member to be developed.

The tables can be conservatively applied when the support thickness ranges from the thickness of the end plate to the thickness of the flange of a W360x79 column. Fabricators would select connection properties listed in the middle of the first page of each table to match shop preferences.

The remaining portion of this chapter describes some of the possible applications for such design tables and a numeric example is given in Appendix C. A brief comparison between tables developed using the LSD and SVS models is given.

9.2 Use by Structural Steel Fabricators

9.2.1. Selection of a flexible end plate connection when the supported member span length and factored load are known. The designer refers to the design tables, eg. Tables 9.1 to 9.4, that contains the correct beam size, correct loading pattern, correct material properties, and then

- i. determines the two beam lengths given in the tables that bracket the given span length under the heading

Beam
Length
(mm)

- ii. for these two span lengths, determines which of the series of tables contains a factored load level closest to the given factored load level in the column near the bottom of the first page of the table with the heading

FACT
LOAD
(kN/m)

- iii. from the second and third pages of the correct table, determines which of the four connection plate thicknesses is preferred, based on any one of several different conditions. In any case the strength ratios given in these pages, corresponding to the bracketing span lengths must exceed a value of 1.00, which indicates the intended margin of safety is provided.

The strength ratios are found under the following column headings:

$$\frac{V_{\text{bolt}}}{V_f} \quad \frac{V_{\text{weld}}}{V_f} \quad \frac{V_{\text{plt.}}}{V_f} \quad \frac{V_{\text{beam}}}{V_f}$$

When selecting a connection for a span which is not close in length to either of the bracketing lengths, selection of a connection detail specified for the shorter of the two spans, which carries a larger total load, results in a detail designed for a shear force greater than the beam in question. The connection associated with the longer span, designed for a larger rotation, will definitely be large enough to prevent bottom flange bearing from occurring. With connection strength assured, the designer may wish to select the preferred connection detail based on:

- a. a preferred plate thickness,
- b. the minimum cost as determined in the table under the table heading:

Conn.
Cost
(\$)

- c. the smallest end moment. The magnitude of the factored level connection moment is listed under the following column headings:

Beam End
Mom. FL
1 (kN/m)

Beam End
Mom. FL
2 (kN/m)

The first column is associated with the recommended connection detail as given in the table, which also happens to be the smallest connection detail possible, and the second column the largest possible connection detail that can fit on the supported member based on the given connection geometry. It is possible for the connection details in both columns to be identical.

In some instances, for certain span lengths all of the connection design information in the tables is replaced with a word or a four letter abbreviation. This occurs when the connection detail is unsatisfactory for the factored load imposed upon it and it serves as a warning to the designer to investigate other solutions. The word or abbreviated word that can occur signifies the problem condition as follows:

Beam- supported member web must attain a stress level in tension greater than $1.14\sigma_{yw}$ at the factored load level,

Bolt- fracture in at least the two top bolts of the connection is likely at the factored load level,

Cope- combined effect of shear and flexure at the coped section of the supported member are likely too large at the factored load level,

Fling- bottom flange of the supported member will likely bear on the support prior to attaining the factored level rotation,

Plat- end plate will likely rupture prior to attaining the factored load level, and

Weld- fillet weld at the top of the connection will likely fracture prior to the factored load level being attained.

These six words or abbreviations, along with one other abbreviation, can also appear in the table under the following column heading:

Governing
Condition

In this column these words are intended to notify the designer which of seven conditions governs the connection design. The words and abbreviations have the following meanings:

Beam- supported member web is the weakest component of the connection,

Bolt- bolt group is the weakest component of the connection,

Cope- indicates that for a coped member, the coped section is weaker than any of the connection components,

Fling- bottom flange of the supported member will bear on the support prior to attaining the factored level rotation and therefore the connection is unacceptable,

Plat- end plate is the weakest component of the connection,

Rote- size of the connection is not governed by the strength of the various components instead the connection size is selected to ensure that bottom flange bearing does not occur, and

Weld- fillet weld group is the weakest component in the connection.

- iv. once the preferred connection detail is selected, the designer records all of the pertinent design information for the connection detail according to the following symbolic system, replacing the symbols with the appropriate numbers from the table:

FEP(Fyp-Dh-g-p-Dend-Dedg-Dtop-tp-L) B(Fub-Db-#)

W(Xu-f)

9.2.2. Selection of a connection detail when only the end shear is known. The problem with this situation is that the effect of connection rotation on the primary shear capacity of the connection is not known directly and must be estimated. This situation may arise in composite construction where it is not possible to arrive at the factored end rotation of the supported member using the CDSSSS program in its existing form. Once the designer has the set of design tables, eg. Tables 9.1 to 9.4, that gives the correct beam size and the correct material properties, the suggested approach for the designer to take is to

- i. assume that the supported member is loaded by a uniform load to 90 or 100 percent of the maximum possible factored level,
- ii. enter page two and page three of the appropriate design table and locate the factored shear force nearest the required value under the following column heading:

FACT
SHEAR
(kN)

- iii. select one of the four possible connection details, as described in 9.2.1.iii above, and
- iv. record connection detail information as in 9.2.1.iv above.

9.2.3. Determination of the necessary dead load camber for a supported member when the span length and service level loading is known. The designer enters the design tables, eg. Tables 9.1 to 9.4, that give the correct beam size, loading pattern, material properties, and then

- i. from page one of any one of the tables, determines the two beam span lengths that bracket the span length of the beam in question in the column with the heading

Beam
Length
(mm)

- ii. for these two span lengths, locates, at the bottom of the first page, the corresponding maximum span deflection for 100 percent dead loading in the column with the heading

Max DL
defl
(mm)

This is the required dead load camber assuming that the entire load is dead. Multiply this value by the ratio of the specified dead load to the 100 percent specified dead load found on page one in the column with the heading

SPEC
100%DL
(kN/m)

The resulting value is the required dead load camber.

Since the dead load deflections used in the above procedure were developed assuming simple supports and ignoring the finite rotational restraint of the connection, the dead load camber calculated herein will be larger than necessary.

9.3 Use by Structural Engineers

9.3.1. Checking connection details specified by fabricator on shop drawings. Either of the procedures described under 9.2.1. and 9.2.2. of Section 9.2 could be applied by the consulting engineer.

9.3.2. Estimation of support moments developed in simple frames. Knowing the loading and span lengths, engineers can investigate the range of support moments likely to exist for various connection details. This information, as described in Section 9.2.1.iii, is available on pages two and three of the design tables. By evaluating the magnitude of end moments which may develop, the consulting engineer can ensure that supporting members are designed adequately. Alternatively, the consulting engineer may choose to specify a range of geometric and material properties for the connection details which would ensure sufficient flexibility.

9.3.3. Evaluation of the deflection serviceability of beams and girders. The designer enters the design tables, eg. Tables 9.1 to 9.4, that give the correct member size, the correct loading pattern, and the correct material properties, and then

- i. from the first page of any one of the tables, determines the two beam lengths given in the tables

that bracket the given span length under the column heading

Beam
Length
(mm)

- ii. determines, at the bottom of the first page, the corresponding magnitude of the maximum span deflection for a 100 percent service dead load, listed under the column heading

Max DL
defl
(mm)

This value is multiplied by the ratio of the given specified load to the 100 percent specified dead load, which is found on page one under the column the heading

SPEC
100%DL
(kN/m)

The resulting value is the maximum service level deflection for the given loading.

- iii. In order to assess the relative magnitude of the member deflection, enter on page one of the table, the column with the heading

$\frac{\text{Length}}{\text{DLdefl}}$

This gives the ratio of the supported member span length to the 100 percent dead load deflection. As before, this value is multiplied by the ratio of the service load to the 100 percent dead load listed in the table in order to determine the relative deflection.

9.3.4. Evaluation of the end rotation serviceability of beams and girders. The movement away from the support of the top surface of the supported member may be of concern to designers when beams are relatively deep, or when the live load is relatively large. To evaluate this movement the designer, once the end detail has been selected,

- i. determines, from page two or three of the appropriate table, the two beam lengths that bracket the given span length, as given under the column heading

Beam
Length
(mm)

- ii. determines, for the correct connection detail, the corresponding magnitude of the horizontal displacement at the top of the supported member due to 100 percent service dead load, from the second row of tabular information. This is listed under the heading

Dend Top
Beam DL
(mm)

This value is multiplied by the ratio of the given specified load to the 100 percent specified dead load, which is found on page one under the column heading

SPEC
100%DL
(kN/m)

This gives an estimation of the maximum service level movement at the top of the beam adjacent to the support.

9.4 Comparison Between LSD and SVS Models

Table 9.5 shows a standardized connection design table identical in all respects to Table 9.2 except that the SVS connection design model was used instead of the LSD model. For the geometric and material properties in these tables it is seen that the two connection design models generally result in equivalent connection sizes except where the span lengths are greater than approximately 15 times the supported member depth. For these longer spans, the LSD table indicates that longer connection plates are required than those indicated in the SVS table. The reason for this is that bottom flange bearing often occurs where spans are relatively long and the LSD model requires that connections be long enough to prevent such bearing.

On page three of these tables, the connections with 12 mm thick end plates tend to be at least one pitch longer in the LSD table than in the SVS table. This is due to the fact

LSD model considers the effect of secondary forces on the primary shear capacity of the connection components. For connections with 12 mm thick plates the secondary forces are significant and therefore the primary shear capacity is reduced considerably. A similar phenomena would likely be observed if the gage of the connection was reduced.

Thus, where connection details are relatively flexible, that is where the plates are relatively thin and the gages are relatively long, and where the supported member span is relatively short, the SVS and LSD models predict similar connection details. Otherwise, the LSD model will likely require connection details larger than the SVS model. However, consistent margins of safety and sufficient unrestrained rotation characteristics will result.

Table 9.1 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 100 % of the Ultimate Load (LSD Model)

(a) Output Page 1

WS20X82		(Available in Canada And U.S.A.). Normally Used As A Beam	
SECTION PROPERTIES		MATERIAL PROPERTIES	
Nominal Mass,	M = 82 Kg/m	Yield Strength,	Fy = 300.0 MPa
Gross Sectional Area,	A = 10500 mm ²	Modulus of Elasticity,	E = 200000 MPa
x-Axis Mom of Inertia,	Ix = 479.0x10 ⁸ mm ⁴	SECTION CLASS & SECTION STRENGTH	
x-Axis Section Modulus,	Sx = 1810.0x10 ³ mm ³	Flange Width/Thickness, b/t = 7.9, CLASS 1	
x-Axis Plastic Modulus,	Ipx = 2070.0x10 ⁸ mm ⁴	Web Width/Thickness, h/w = 52.8, CLASS 1B	
y-Axis Mom of Inertia,	Iy = 20.3x10 ⁸ mm ⁴	Resistance Factor,	$\phi = 0.90$
y-Axis Section Modulus,	Sy = 184.0x10 ³ mm ³	Fact. Yld. Moment Cap.,	My = 488.7 kNm
y-Axis Plastic Modulus,	Ipy = 303.0x10 ⁸ mm ⁴	Fact. Moment Capacity,	Mp = 558.8 kNm
Torsional Constant,	J = 830.0x10 ³ mm ⁴	Fact. Shear Capacity,	Vp = 893.8 kN
Warping Constant,	Cw = 1240.0x10 ⁹ mm ⁶	Max. Shear Strength,	Fs = 188.0 MPa
Overall Section Depth,	d = 828.0 mm		$\phi = 0.85F_y$
Flange Width,	b = 208.0 mm		
Flange Thickness,	t = 13.3 mm		
Clear Flange Distance,	FT = 85.5 mm		
Web Height,	h = 501.4 mm		
Web Thickness,	w = 8.5 mm		
Clear Web Distance,	WT = 470.0 mm		

FLEXIBLE END PLATE CONNECTION		FEP(Fyp-Dh-g-p-Bend-Deag-Stop-tp-L) E(Fub-DD-W) W(RU-F)	
PLATE PROPERTIES		BOLT PROPERTIES	
Yield Strength,	Fyp = 300.0 MPa	Ultimate Strength,	Fub = 825.0 MPa
Ultimate Strength,	Fup = 450.0 MPa	Resistance Factor,	$\phi_b = 0.87$
Modulus of Elasticity,	Ep = 200000.0 MPa	Nominal Diameter,	Db = 18.0 mm
Resistance Factor,	$\phi_p = 0.90$	Fact. Tensile Cap.,	Trb = 117.8 kN/Bolt
		Fact. Shear Capacity,	Vrb = 85.8 kN/Bolt
Bolt Hole Diameter,	Dh = 31.0 mm	Bolt Cost,	COSTB = \$ 1.00/Bolt
Bolt Hole Gage,	g = 140.0 mm	WELD PROPERTIES	
Bolt Hole Pitch,	p = 75.0 mm	Ultimate Strength,	Su = 450.0 MPa
Bolt Hole End Dist.,	Bend = 35.0 mm	Resistance Factor,	$\phi_w = 0.87$
Bolt Hole Edge Dist.,	Deag = 30.0 mm	Fillet Weld Leg Size,	fw = 5.0 mm
Bolt Hole Top Dist.,	Stop = 75.0 mm	Fact. Long Sh. Cap.,	Vpw = 0.78 kN/mm
Material Cost,	COSTP = \$ 1.00/kg	Fillet Weld Cost,	COSTW = \$ 0.10/mm

UNIFORMLY DISTRIBUTED LOAD @ 100 % OF THE ULTIMATE LOAD LEVEL FOR THIS MEMBER												
#	Beam Length (mm)	Length Depth	FACT LOAD (kN/m)	SPEC 100%DL (kN/m)	Vf	Nr	Max PL Defl (mm)	Max DL Defl (mm)	Length FLDLFT	Length DLDLFT	Bend PL (PAGE)	Bend DL (PAGE)
1	1584	3.0	1128.8	802.8	1.0	1.9	1.0	0.8	1540	2060	0.0020	0.0016
2	3168	6.0	448.8	358.4	1.3	1.0	30.5	4.9	158	845	0.0187	0.0048
3	4752	9.0	198.0	158.4	1.9	1.0	48.8	11.0	98	433	0.0263	0.0074
4	8338	12.0	111.4	89.1	2.9	1.0	82.8	19.9	68	325	0.0383	0.0098
5	7930	15.0	71.3	57.0	3.3	1.0	147.0	30.5	54	260	0.0488	0.0123
6	5504	18.0	48.8	39.8	3.8	1.0	314.1	43.9	44	216	0.0582	0.0148
7	11088	21.0	38.4	29.1	4.4	1.0	280.4	58.9	40	188	0.0631	0.0173
8	12672	24.0	37.8	22.3	5.1	1.0	371.2	78.1	34	162	0.0728	0.0197
9	14256	27.0	23.0	17.8	5.7	1.0	474.0	98.8	30	144	0.0821	0.0222
10	15840	30.0	17.8	14.3	6.3	1.0	589.4	122.0	27	120	0.0918	0.0248

Table 9.1 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 100 % of the Ultimate Load (LSD Model)

(b) Output Page 2

W30X82 (Available in Canada And U.S.A.), Normally Used As A Beam											
CONN #1 : PEP(300-21-140- 75-35-30- 75- 8-L) B: 825-19-W) W(480- 5) LSD											
Support. Flange of a W30X79 Column, 18.8 mm thick. Material Prop. as per Supported Member											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VV	Vweld VV	Vplt VV	Vbeam VV
1	1884	3.0	445.0	12	213.18	Weld	893.8	0.85	0.82	0.87	0.86
2	3188	8.0	445.0	12	213.18	Weld	705.7	1.08	1.04	1.10	1.05
3	4752	9.0	445.0	12	213.18	Plate	470.5	1.81	1.56	1.55	1.53
4	8336	12.0	445.0	12	213.18	Plate	352.8	2.04	1.89	2.94	1.83
5	7820	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
6	8504	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
7	11088	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
8	12672	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
9	14256	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
10	15840	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
#	Beam Top P11 FL (mm)	Beam Top Beam, SL (mm)	Beam End Mem. FL 1 (kNm)	Beam End Mem. FL 2 (kNm)	Hand1 Mr	Hand2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	0.7	0.6	23.9	23.9	0.04	0.04	1128.8	1128.8			
2	8.5	2.0	38.3	38.3	0.07	0.07	476.1	476.1			
3	10.3	2.1	47.0	47.0	0.08	0.08	214.7	214.7			
4	18.2	8.2	58.0	58.0	0.10	0.10	122.5	122.5			
5	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
6	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
7	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
8	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
9	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
10	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			

CONN #2 : PEP(300-21-140- 75-35-30- 75- 8-L) B: 825-19-W) W(480- 5) LSD											
Support. Flange of a W36X79 Column, 18.8 mm thick. Material Prop. as per Supported Member.											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VV	Vweld VV	Vplt VV	Vbeam VV
1	1884	3.0	445.0	12	214.57	Weld	893.8	0.88	0.78	1.18	0.84
2	3188	8.0	445.0	12	214.57	Weld	705.7	1.12	0.98	1.47	0.99
3	4752	9.0	370.0	10	178.84	Weld	470.5	1.40	1.22	1.83	1.23
4	8336	12.0	445.0	12	214.57	Plate	352.8	2.04	1.89	2.94	1.83
5	7820	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
6	8504	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
7	11088	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
8	12672	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
9	14256	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
10	15840	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling

#	Beam Top P11 FL (mm)	Beam Top Beam, SL (mm)	Beam End Mem. FL 1 (kNm)	Beam End Mem. FL 2 (kNm)	Hand1 Mr	Hand2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)	
1	0.6	0.6	38.8	38.8	0.07	0.07	1128.8	1128.8	
2	8.1	1.8	57.8	57.8	0.10	0.10	481.4	481.4	
3	8.2	2.4	43.8	43.8	0.08	0.12	213.5	231.5	
4	13.3	3.8	75.8	75.8	0.14	0.14	128.4	128.4	
5	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	
6	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	
7	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	
8	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	
9	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	
10	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	

Table 9.1 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 100 % of the Ultimate Load (LSD Model)

(c) Output Page 3

W36x82 (Available in Canada And U.S.A.), Normally Used As A Beam											
CONN #3 : FEP(300-21-140- 75-35-30- 75-10-L) B(825-19-F) W(480- 5) LSD											
Support: Flange of a W36x75 Column: 15 & mm thick Material Prop. as per Supported Member Supported member is uncoped											
#	Beam Length (mm)	Length Depth	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VV	Vweld VV	Vplt VV	Vbeam VV
1	1584	3.0	445.0	12	215.97	Weld	893.8	0.82	0.83	1.45	0.81
2	3168	8.0	445.0	12	215.97	Weld	705.7	1.00	0.75	1.84	0.81
3	4752	9.0	445.0	12	215.97	Weld	470.5	1.48	1.10	2.78	1.29
4	6336	12.0	445.0	12	215.97	Plate	352.8	1.87	1.37	3.88	1.59
5	7920	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt
6	9504	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
7	11088	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
8	12672	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
9	14256	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
10	15840	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
#	Dend Top Flt FL (mm)	Dend Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 MR	Mend2 MR	Adjusted wFL 1 (kN/m)	Adjusted wFL 2 (kN/m)			
1	0.5	0.5	57.5	57.5	0.10	0.10	1128.8	1128.8			
2	5.8	1.7	79.7	79.7	0.14	0.14	509.0	509.0			
3	8.9	2.8	88.1	88.1	0.18	0.18	229.2	229.2			
4	12.6	3.6	98.0	98.0	0.17	0.17	130.5	130.5			
5	15.9	4.6	103.5	103.5	0.19	0.19	84.5	84.5			
6	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
7	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
8	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
9	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
10	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			

CONN #4 : FEP(300-21-140- 75-35-30- 75-12-L) B(825-19-F) W(480- 5) LSD											
Support: Flange of a W36x75 Column: 15 & mm thick Material Prop. as per Supported Member Supported member is uncoped											
#	Beam Length (mm)	Length Depth	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VV	Vweld VV	Vplt VV	Vbeam VV
1	1584	3.0	445.0	12	217.35	Weld	893.8	0.83	0.42	1.74	0.79
2	3168	8.0	445.0	12	217.35	Weld	705.7	0.72	0.38	2.21	0.80
3	4752	9.0	445.0	12	217.35	Weld	470.5	0.99	0.53	3.31	1.18
4	6336	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld
5	7920	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt
6	9504	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
7	11088	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
8	12672	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
9	14256	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
10	15840	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling
#	Dend Top Flt FL (mm)	Dend Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 MR	Mend2 MR	Adjusted wFL 1 (kN/m)	Adjusted wFL 2 (kN/m)			
1	0.5	0.4	73.9	73.9	0.13	0.13	1128.8	1128.8			
2	5.2	1.8	103.4	103.4	0.18	0.18	527.9	527.9			
3	8.4	2.4	111.1	111.1	0.20	0.20	237.4	237.4			
4	11.8	3.3	119.1	119.1	0.21	0.21	135.1	135.1			
5	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt	Bolt			
6	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
7	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
8	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
9	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			
10	Fling	Fling	Fling	Fling	Fling	Fling	Fling	Fling			

Table 9.2 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 95 % of the Ultimate Load (LSD Model)

(a) Output Page 1

W30x222		(Available in Canada And U.S.A.), Normally Used As A Beam.	
SECTION PROPERTIES		MATERIAL PROPERTIES	
Nominal Mass,	m = 82 Kg/m	Yield Strength,	Fy = 300.0 MPa
Gross Sectional Area,	A = 10500 mm ²	Modulus of Elasticity,	E = 200000 MPa
x-Axis Mom. of Inertia,	Ix = 478.0x10 ⁸ mm ⁴	SECTION CLASS & SECTION STRENGTH	
x-Axis Section Modulus,	Sx = 1810.0x10 ³ mm ³	Flange Width/Thickness, b/t = 7.8, CLASS 1	
x-Axis Plastic Modulus,	Ixp = 2070.0x10 ³ mm ³	Web Width/Thickness, h/w = 52.8, CLASS 1B	
y-Axis Mom. of Inertia,	Iy = 20.3x10 ⁸ mm ⁴	Resistance Factor,	φ = 0.90
y-Axis Section Modulus,	Sy = 194.0x10 ³ mm ³	Fact. Yld. Moment Cap.,	My = 488.7 kNm
y-Axis Plastic Modulus,	Iyp = 303.0x10 ³ mm ³	Fact. Moment Capacity,	Mr = 558.9 kNm
Torsional Constant,	J = 530.0x10 ³ mm ⁴	Fact. Shear Capacity,	Vr = 893.9 kN
Warping Constant,	Cw = 1340.0x10 ⁸ mm ⁶	Max. Shear Strength,	Fs = 198.0 MPa
Overall Section Depth,	d = 528.0 mm		φ 85xPy
Flange Width,	b = 209.0 mm		
Flange Thickness,	t = 13.3 mm		
Clear Flange Distance,	ft = 85.5 mm		
Web Height,	h = 501.4 mm		
Web Thickness,	w = 8.5 mm		
Clear Web Distance,	wt = 470.0 mm		

FLEXIBLE END PLATE CONNECTION		FEP(Fyp-Dh-g-p-Bend-Dedg-Dtop-tp-L) B(Fub-Db-w) W(Xu-F)	
PLATE PROPERTIES		BOLT PROPERTIES	
Yield Strength,	Fyp = 300.0 MPa	Ultimate Strength,	Fub = 825.0 MPa
Ultimate Strength,	Fup = 450.0 MPa	Resistance Factor,	φb = 0.87
Modulus of Elasticity,	Ep = 200000.0 MPa	Nominal Diameter,	Db = 19.0 mm
Resistance Factor,	φp = 0.90	Fact. Tensile Cap.,	Trb = 117.5 kN/Bolt
Bolt Hole Diameter,	Dh = 21.0 mm	Fact. Shear Capacity,	Vrb = 66.8 kN/Bolt
Bolt Hole Gauge,	g = 140.0 mm	Bolt Cost,	COSTB = \$ 10.00/Bolt
Bolt Hole Pitch,	p = 75.0 mm	WELD PROPERTIES	
Bolt Hole End Dist.,	Bend = 35.0 mm	Ultimate Strength,	Xu = 480.0 MPa
Bolt Hole Edge Dist.,	Dedg = 30.0 mm	Resistance Factor,	φw = 0.87
Bolt Hole Top Dist.,	Dtop = 75.0 mm	Pillet Weld Leg Size,	f = 5.0 mm
Material Cost,	COSTP = \$ 1.00/kg	Fact. Long. Sh. Cap.,	Vrw = 0.75 kN/mm
		Pillet Weld Cost,	COSTW = \$ 0.10/mm

UNIFORMLY DISTRIBUTED LOAD @ 95. % of the ULTIMATE LOAD LEVEL for this member.												
#	Beam Length (mm)	Length Depth (mm)	FACT LOAD (kN/m)	SPEC 100%DL (kN/m)	Vr Vt	Mr Mt	Max PL Defl (mm)	Max DL Defl (mm)	Length FLOEFT	Length DLOEFT	θend PL (rads)	θend DL (rads)
1	1584	3.0	1072.2	857.7	1.1	1.7	0.9	0.7	1727	2158	0.0019	0.0015
2	3168	6.0	423.2	338.8	1.3	1.1	8.5	6.8	488	883	0.0085	0.0047
3	4752	9.0	185.1	150.5	2.0	1.1	15.5	10.4	308	486	0.0101	0.0070
4	6336	12.0	105.8	84.6	3.7	1.1	25.3	16.8	224	342	0.0138	0.0096
5	7920	15.0	67.7	54.3	3.3	1.1	44.9	29.0	178	273	0.0171	0.0117
6	9504	18.0	47.0	37.6	4.0	1.1	65.2	41.7	148	228	0.0208	0.0140
7	11088	21.0	34.6	27.6	4.7	1.1	85.5	56.8	130	185	0.0238	0.0164
8	12672	24.0	28.5	21.2	5.3	1.1	113.1	74.2	112	171	0.0272	0.0187
9	14256	27.0	20.9	16.7	6.0	1.1	144.3	93.9	99	162	0.0307	0.0211
10	15840	30.0	16.9	13.5	6.7	1.1	179.7	118.9	88	137	0.0342	0.0234

Table 9.2 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 95 % of the Ultimate Load (LSD Model)

(b) Output Page 2

W30X82 (Available In Canada And U.S.A.), Normally Used As A Beam											
CONN #1 : FEP(300-21-140- 75-35-30- 75- 8-L) B: 825-19-W: W480- 5) LSD											
Support: Flange of a W30X79 Column, 16.8 mm thick Material Prop. as per Supported Member											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VF	Vweld VF	Vplt VF	Vbeam VF
1	1584	3.0	445.0	12	213.18	Weld	849.2	0.89	0.87	0.92	0.89
2	3168	5.0	445.0	12	213.18	Weld	870.4	1.13	1.10	1.18	1.11
3	4752	9.0	295.0	8	141.77	Weld	448.9	1.13	1.08	1.16	1.12
4	6336	12.0	295.0	8	141.77	Weld	335.2	1.51	1.45	1.54	1.50
5	7920	15.0	295.0	8	141.77	Rate	268.2	1.89	1.81	1.92	1.88
6	9504	18.0	370.0	10	177.48	Rate	223.5	2.63	2.73	2.80	2.75
7	11088	21.0	370.0	10	177.48	Rate	181.5	3.31	3.18	3.38	3.19
8	12672	24.0	445.0	12	213.18	Rate	167.5	4.83	4.39	4.85	4.30
9	14256	27.0	445.0	12	213.18	Rate	149.0	5.18	4.82	5.23	4.78
10	15840	30.0	445.0	12	213.18	Rate	134.1	5.67	5.44	5.81	5.28

#	Dend Top Plt. FL (mm)	Dend Top Beam. DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 Mr	Mend2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)
1	0.6	0.8	23.3	23.3	0.04	0.04	1128.8	1128.8
2	2.4	1.9	31.2	31.2	0.08	0.08	448.1	448.1
3	2.8	2.0	14.5	33.4	0.03	0.06	193.2	189.9
4	3.5	2.8	15.1	35.7	0.03	0.06	108.8	112.9
5	4.4	3.5	15.9	38.7	0.03	0.07	88.7	72.7
6	5.1	5.1	27.5	41.8	0.05	0.07	49.5	50.7
7	7.7	5.0	29.1	44.8	0.05	0.08	38.4	37.5
8	10.8	8.1	47.8	47.8	0.09	0.09	28.8	28.8
9	12.0	9.1	51.0	51.0	0.09	0.09	22.9	22.9
10	13.4	10.1	54.1	54.1	0.10	0.10	18.7	18.7

CONN #2 : FEP(300-21-140- 75-35-30- 75- 8-L) B: 825-19-W: W480- 5) LSD											
Support: Flange of a W30X79 Column, 16.8 mm thick Material Prop. as per Supported Member											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VF	Vweld VF	Vplt VF	Vbeam VF
1	1584	3.0	445.0	12	214.57	Weld	849.2	0.93	0.82	1.22	0.89
2	3168	5.0	445.0	12	214.57	Weld	870.4	1.18	1.04	1.55	1.08
3	4752	9.0	295.0	8	142.70	Weld	448.9	1.18	1.03	1.54	1.09
4	6336	12.0	295.0	8	142.70	Weld	335.2	1.57	1.37	2.05	1.45
5	7920	15.0	220.0	8	106.78	Weld	268.2	1.47	1.28	1.91	1.37
6	9504	18.0	295.0	8	142.70	Rate	223.5	2.36	2.08	3.08	2.13
7	11088	21.0	370.0	10	178.84	Rate	181.5	3.44	3.01	4.51	2.95
8	12672	24.0	370.0	10	178.84	Rate	167.5	3.93	3.42	5.15	3.44
9	14256	27.0	445.0	12	214.57	Rate	149.0	5.18	4.56	6.97	4.41
10	15840	30.0	445.0	12	214.57	Rate	134.1	5.63	5.01	7.74	4.85

#	Dend Top Plt. FL (mm)	Dend Top Beam. DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 Mr	Mend2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)
1	0.6	0.5	38.7	38.7	0.07	0.07	1128.8	1128.8
2	2.3	1.8	50.0	50.0	0.09	0.09	483.1	483.1
3	2.4	1.8	23.2	52.8	0.04	0.08	196.3	208.8
4	3.3	2.8	24.0	55.8	0.04	0.10	110.8	118.9
5	3.1	2.5	18.1	58.0	0.03	0.10	89.9	75.1
6	5.0	4.0	25.9	51.1	0.05	0.11	49.3	52.4
7	7.2	5.5	42.8	53.9	0.08	0.11	37.3	38.7
8	8.4	8.4	44.4	67.2	0.08	0.12	28.7	29.8
9	11.2	8.5	70.4	70.4	0.13	0.13	23.7	23.7
10	12.5	8.5	73.7	73.7	0.13	0.13	19.3	19.3

Table 9.2 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 95 % of the Ultimate Load (LSD Model)

(c) Output Page 3

W30X182 (Available In Canada And U.S.A. Normally Used As A Beam)											
CONN #3 : FEP(300-21-140- 75-35-30- 75-10-L) S(825-19-W) W(480- S)								LSD			
Support, Flange of a W30X182 Column, 18.8 mm thick Material Prop. as per Supported Member. Supported member is uncoped.											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vplt V _f	Vbeam V _f
1	1584	3.0	445.0	12	215.97	Weld	849.2	0.85	0.85	1.83	0.85
2	3168	8.0	445.0	12	215.97	Weld	870.4	1.09	0.84	1.94	1.03
3	4752	9.0	370.0	10	178.75	Weld	446.9	1.34	1.01	2.41	1.27
4	6336	12.0	295.0	8	142.62	Weld	335.2	1.44	1.08	2.87	1.35
5	7920	15.0	220.0	8	107.45	Weld	288.2	1.38	1.04	2.39	1.30
6	9504	18.0	295.0	8	142.62	Rate	223.5	2.15	1.82	3.85	1.98
7	11088	21.0	295.0	8	142.62	Rate	191.8	2.50	1.89	4.49	2.31
8	12672	24.0	370.0	10	178.75	Rate	167.8	3.48	2.53	5.44	3.14
9	14256	27.0	370.0	10	178.75	Rate	148.0	3.88	2.83	7.24	3.48
10	15840	30.0	445.0	12	215.97	Rate	134.1	4.81	3.88	9.48	4.49

#	Dend Top Plt. FL (mm)	Dend Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 Mr	Mend2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)	
1	0.5	0.4	58.0	58.0	0.10	0.10	1128.8	1128.8	
2	2.0	1.6	71.5	71.5	0.13	0.13	480.2	480.2	
3	2.7	2.1	75.4	75.4	0.09	0.13	208.5	214.8	
4	3.0	2.4	34.3	77.9	0.08	0.14	112.8	121.3	
5	2.9	2.4	18.8	80.1	0.04	0.14	70.2	77.9	
6	4.8	3.8	38.1	83.1	0.08	0.15	80.2	84.4	
7	5.4	4.3	38.5	85.8	0.07	0.15	35.9	40.1	
8	7.8	6.0	55.7	88.9	0.11	0.16	29.4	30.9	
9	8.9	6.8	81.5	91.2	0.11	0.16	23.3	24.9	
10	11.9	8.9	94.2	94.2	0.17	0.17	19.8	19.8	

CONN #4 : FEP(300-21-140- 75-35-30- 75-12-L) S(825-19-W) W(480- S) LSD											
Support, Flange of a W30x75 Column, 18.8 mm thick. Material Prop. as per Supported Member. Supported member is uncoped.											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vplt V _f	Vbeam V _f
1	1584	3.0	445.0	12	217.38	Weld	849.2	0.89	0.44	1.83	0.83
2	3168	8.0	445.0	12	217.38	Weld	870.4	0.77	0.40	2.32	0.86
3	4752	9.0	445.0	12	217.38	Weld	446.9	1.15	0.80	3.48	1.34
4	6336	12.0	445.0	12	217.38	Weld	335.2	1.53	0.80	4.85	1.72
5	7920	15.0	445.0	12	217.38	Weld	288.2	1.89	1.00	5.81	2.11
6	9504	18.0	370.0	10	180.95	Weld	223.5	2.00	1.15	5.78	2.12
7	11088	21.0	370.0	10	180.95	Weld	191.8	2.31	1.33	5.75	2.42
8	12672	24.0	445.0	12	217.38	Weld	167.8	2.73	1.45	9.29	3.24
9	14256	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld
10	15840	30.0	370.0	10	180.95	Rate	134.1	2.41	1.02	9.88	3.38

#	Dend Top Plt. FL (mm)	Dend Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 Mr	Mend2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)	
1	0.4	0.4	72.2	72.2	0.13	0.13	1128.8	1128.8	
2	1.9	1.4	92.9	92.9	0.17	0.17	497.2	497.2	
3	3.0	2.3	98.2	98.2	0.18	0.18	222.9	222.9	
4	4.1	3.1	101.4	101.4	0.18	0.18	128.0	128.0	
5	5.3	4.0	103.8	103.8	0.18	0.18	81.0	81.0	
6	5.3	4.1	73.7	107.1	0.13	0.19	53.6	58.5	
7	6.1	4.8	75.4	109.8	0.13	0.20	39.5	41.7	
8	8.7	6.8	111.8	111.8	0.20	0.20	32.0	32.0	
9	Weld	Weld	Weld	Weld	Weld	Weld	Weld	Weld	
10	8.1	7.0	80.1	117.5	0.14	0.21	19.8	20.7	

Table 9.3 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 90 % of the Ultimate Load (LSD Model)

(a) Output Page 1

W30x82		Available in Canada And U.S.A. Normally Used As A Beam	
SECTION PROPERTIES		MATERIAL PROPERTIES	
Nominal Mass,	M = 82 kg/m	Yield Strength,	Fy = 300.0 MPa
Cross Sectional Area,	A = 10500 mm ²	Modulus of Elasticity,	E = 200000 MPa
x-Axis Mom. of Inertia,	Ix = 479 0x10 ⁸ mm ⁴	SECTION CLASS & SECTION STRENGTH	
x-Axis Section Modulus,	Sx = 1810 0x10 ³ mm ³	Flange Width/Thickness, B/t = 7.8,	CLASS 1
x-Axis Plastic Modulus,	Zx = 2070 0x10 ³ mm ³	Web Width/Thickness, H/w = 52.8,	CLASS 1B
y-Axis Mom. of Inertia,	Iy = 20 3x10 ⁸ mm ⁴		CLASS 1C
y-Axis Section Modulus,	Sy = 184 0x10 ³ mm ³	Resistance Factor,	$\phi = 0.90$
y-Axis Plastic Modulus,	Zy = 303 0x10 ³ mm ³	Fact. Yld. Moment Cap.,	My = 488.7 kNm
Torsional Constant,	J = 530 0x10 ³ mm ⁴	Fact. Moment Capacity,	Mp = 558.9 kNm
Warping Constant,	Cw = 1240 0x10 ⁸ mm ⁶	Fact. Shear Capacity,	Vr = 893.9 kN
Overall Section Depth,	D = 528.0 mm	Max. Shear Strength,	Ps = 198.0 MPa
Flange Width,	B = 209.0 mm		$\phi = 0.85 F_y$
Flange Thickness,	t = 13.3 mm		
Clear Flange Distance,	FT = 85.9 mm		
Web Height,	H = 501.4 mm		
Web Thickness,	w = 9.5 mm		
Clear Web Distance,	WT = 470.0 mm		

FLEXIBLE END PLATE CONNECTION		FEP: Fyp-DH-g-p-Bend-Bedg-Dtop-tp-L) B(Fub-Dp-#) W(Lu-F)	
PLATE PROPERTIES		BOLT PROPERTIES	
Yield Strength,	Fyp = 300.0 MPa	Ultimate Strength,	Fub = 425.0 MPa
Ultimate Strength,	Fup = 480.0 MPa	Resistance Factor,	$\phi_b = 0.87$
Modulus of Elasticity,	Ep = 200000.0 MPa	Nominal Diameter,	DB = 19.0 mm
Resistance Factor,	$\phi_p = 0.90$	Fact. Tensile Cap.,	TpD = 117.5 kN/Bolt
Bolt Hole Diameter,	DH = 21.0 mm	Fact. Shear Capacity,	VrD = 85.8 kN/Bolt
Bolt Hole Edge,	g = 140.0 mm	Bolt Cost,	COSTB = \$ 10.00/Bolt
Bolt Hole Pitch,	p = 75.0 mm	WELD PROPERTIES	
Bolt Hole End Dist.,	Bend = 35.0 mm	Ultimate Strength,	Fu = 480.0 MPa
Bolt Hole Edge Dist.,	Bedg = 30.0 mm	Resistance Factor,	$\phi_w = 0.87$
Bolt Hole Top Dist.,	Dtop = 75.0 mm	Fillet Weld Leg Size,	$\phi_w = 5.0$ mm
Material Cost,	COSTP = \$ 1.00/kg	Fact. Long Sh. Cap.,	Vrw = 0.78 kN/mm
		Fillet Weld Cost,	COSTW = \$ 0.10/mm

UNIFORMLY DISTRIBUTED LOAD @ 90 % of the ULTIMATE LOAD LEVEL for this member												
#	Beam Length (mm)	Length Depth	FACT LOAD (kN/m)	SPEC 100TDL (kN/m)	Vr VF	Mr MF	Max FL Defl (mm)	Max DL Defl (mm)	Length FLdefl	Length DLdefl	End FL (rads)	End DL (rads)
1	1584	3.0	1015.7	812.8	1.1	1.8	0.9	0.7	1823.	2278.	0.0018	0.0014
2	3188	9.0	401.0	320.8	1.4	1.1	5.2	4.4	810.	721.	0.0056	0.0044
3	4752	9.0	178.2	142.8	2.1	1.1	12.2	9.9	389.	481.	0.0084	0.0067
4	6336	12.0	100.2	80.2	2.8	1.1	22.3	17.8	285.	361.	0.0113	0.0089
5	7920	15.0	84.2	61.3	3.8	1.1	35.3	27.4	225.	289.	0.0142	0.0111
6	9504	18.0	44.8	35.8	4.2	1.1	51.2	39.5	188.	240.	0.0170	0.0133
7	11088	21.0	32.7	25.2	4.9	1.1	67.8	53.8	184.	208.	0.0197	0.0155
8	12672	24.0	25.1	20.0	5.6	1.1	89.1	70.3	142.	160.	0.0226	0.0177
9	14256	27.0	19.8	15.8	6.3	1.1	113.7	88.9	125.	180.	0.0254	0.0200
10	15840	30.0	18.0	12.8	7.0	1.1	141.1	109.8	112.	144.	0.0283	0.0222

Table 9.3 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 90 % of the Ultimate Load (LSD Model)

(b) Output Page 2

W30x92 Available in Canada And U.S.A. Normally Used As A Beam											
CONN #1 FEP:300-21-140- 75-35-30- 75- 8-L) B(825-19-W) W(480- 5) LSD											
Support, Flange of a W30x75 Column, 16 mm thick Material Prop. as per Supported Member. Supported member is uncoped.											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vp1 V _f	Vbeam V _f
1	1884	3.0	445.0	12	213.18	Weld	804.5	0.94	0.92	0.97	0.94
2	3158	8.0	445.0	12	213.18	Weld	835.1	1.20	1.18	1.23	1.18
3	4752	9.0	295.0	8	141.77	Weld	423.4	1.20	1.14	1.22	1.19
4	6336	12.0	220.0	8	108.07	Weld	317.8	1.20	1.17	1.21	1.20
5	7920	15.0	295.0	8	141.77	Rate	254.0	1.99	1.91	2.03	1.97
6	9504	18.0	295.0	8	141.77	Rate	211.7	2.39	2.29	2.44	2.38
7	11088	21.0	370.0	10	177.48	Rate	181.5	3.49	3.37	3.57	3.39
8	12672	24.0	370.0	10	177.48	Rate	158.8	3.99	3.88	4.08	3.88
9	14256	27.0	445.0	12	213.18	Rate	141.1	5.38	5.21	5.52	5.12
10	15840	30.0	445.0	12	213.18	Rate	127.0	5.98	5.78	6.13	5.89
#	Send Top Plt. FL (mm)	Send Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 Mr	Mend2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	0.8	0.5	22.8	22.8	0.04	0.04	1087.9	1087.9			
2	2.0	1.8	30.2	30.2	0.05	0.05	425.0	425.0			
3	3.1	1.9	14.0	32.1	0.02	0.08	183.2	188.5			
4	2.2	2.0	8.1	34.3	0.01	0.08	101.9	107.1			
5	3.7	3.3	15.2	38.2	0.03	0.08	88.1	88.8			
6	4.4	4.0	15.9	32.7	0.03	0.07	48.0	48.0			
7	5.4	5.7	27.0	41.0	0.05	0.07	34.9	35.4			
8	7.3	8.5	28.5	43.8	0.05	0.08	26.5	27.2			
9	10.0	8.9	48.3	48.3	0.08	0.08	21.9	21.8			
10	11.1	9.5	48.9	48.9	0.09	0.09	17.8	17.8			

CONN #2 FEP:300-21-140- 75-35-30- 75- 8-L) B(825-19-W) W(480- 5) LSD											
Support, Flange of a W30x75 Column, 16 mm thick Material Prop. as per Supported Member. Supported member is uncoped.											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vp1 V _f	Vbeam V _f
1	1884	3.0	445.0	12	214.57	Weld	804.5	0.93	0.93	1.29	0.93
2	3158	8.0	445.0	12	214.57	Weld	835.1	1.24	1.10	1.93	1.14
3	4752	9.0	295.0	8	142.70	Weld	423.4	1.24	1.09	1.83	1.18
4	6336	12.0	220.0	8	108.78	Weld	317.8	1.24	1.09	1.83	1.17
5	7920	15.0	220.0	8	108.78	Weld	284.0	1.99	1.35	2.02	1.49
6	9504	18.0	220.0	8	108.78	Rate	211.7	1.87	1.82	2.42	1.73
7	11088	21.0	295.0	8	142.70	Rate	181.5	2.90	2.84	3.79	2.83
8	12672	24.0	370.0	10	178.84	Rate	158.8	4.19	3.83	5.44	3.94
9	14256	27.0	370.0	10	178.84	Rate	141.1	4.88	4.07	5.12	4.02
10	15840	30.0	370.0	10	178.84	Rate	127.0	5.18	4.50	5.80	4.92
#	Send Top Plt. FL (mm)	Send Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 Mr	Mend2 Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	0.8	0.5	37.8	37.8	0.07	0.07	1128.8	1128.8			
2	1.9	1.8	49.4	49.4	0.09	0.09	440.3	440.3			
3	1.9	1.8	22.8	52.0	0.04	0.09	188.3	188.8			
4	2.0	1.8	13.4	53.8	0.02	0.10	102.9	111.0			
5	2.8	2.4	13.6	55.8	0.03	0.10	85.5	71.2			
6	3.1	2.9	14.1	57.9	0.03	0.10	45.8	49.7			
7	4.8	4.4	28.8	60.2	0.05	0.11	34.4	38.7			
8	6.8	6.1	42.2	62.9	0.08	0.11	27.2	28.2			
9	7.7	6.9	43.8	65.8	0.08	0.12	21.6	22.4			
10	8.8	7.8	48.0	68.3	0.08	0.12	17.8	18.2			

Table 9.3 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 90 % of the Ultimate Load (LSD Model)

(c) Output Page 3

W30x82 (Available In Canada And U.S.A. - Normally Used As A Beam)											
CONN #3 FEP:300-21-140- 75-35-30- 75-10-L1 S1 825-19-W W480- S1											LSD
Support: Flange of a W30x75 Column: 16 mm thick Material Prop. as per Supported Member											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vplt V _f	Vbeam V _f
1	1584	3 0	445 0	12	215 97	Weld	804 5	0 91	0 70	1 81	0 91
2	3168	8 0	445 0	12	215 97	Weld	835 1	1 15	0 89	2 04	1 09
3	4752	8 0	370 0	10	179 79	Weld	423 4	1 41	1 08	2 55	1 38
4	6336	12 0	285 0	8	143 82	Weld	317 6	1 52	1 15	2 71	1 45
5	7920	15 0	220 0	8	107 45	Weld	254 0	1 44	1 10	2 55	1 38
6	9504	18 0	220 0	8	107 45	Weld	211 7	1 73	1 32	3 03	1 55
7	11088	21 0	220 0	8	107 45	Weld	181 5	2 01	1 53	3 54	1 81
8	12672	24 0	285 0	8	143 82	Weld	158 8	2 02	2 28	5 42	2 74
9	14256	27 0	285 0	8	143 82	Weld	141 1	3 39	2 58	8 09	3 10
10	15840	30 0	370 0	10	179 79	Weld	127 0	4 57	3 45	8 49	4 12

#	Dist Top P1t FL (mm)	Dist Top Beam, DL (mm)	Beam End Mom FL 1 (kNm)	Beam End Mom FL 2 (kNm)	Hand1 MP	Hand2 MP	Adjusted wFL 1 (kN/m)	Adjusted wFL 2 (kN/m)	
1	0 5	0 4	54 7	54 7	0 10	0 10	1128 5	1128 5	
2	1 7	1 5	70 5	70 5	0 13	0 13	457 2	457 2	
3	2 3	2 0	50 6	75 2	0 09	0 13	198 1	204 2	
4	2 4	2 2	33 9	75 8	0 08	0 14	107 0	115 2	
5	2 3	2 2	19 5	78 4	0 04	0 14	55 8	74 2	
6	2 9	2 7	19 5	80 0	0 04	0 14	45 3	51 9	
7	3 2	3 2	20 3	82 3	0 04	0 15	34 1	38 1	
8	5 1	4 7	35 7	84 8	0 07	0 15	25 9	29 3	
9	5 9	5 3	37 0	87 4	0 07	0 15	21 3	23 2	
10	8 2	7 1	50 3	85 9	0 11	0 15	15 0	15 8	

CONN #4 FEP:300-21-140- 75-35-30- 75-12-L1 S1 825-19-W W480- S1										LSD	
Support: Flange of a W30x75 Column: 16 mm thick Material Prop. as per Supported Member											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vplt V _f	Vbeam V _f
1	1584	3 0	445 0	12	217 38	Weld	804 5	0 73	0 48	1 94	0 98
2	3168	8 0	445 0	12	217 38	Weld	835 1	0 81	0 43	2 45	1 02
3	4752	8 0	445 0	12	217 38	Weld	423 4	1 32	0 84	3 84	1 48
4	6336	12 0	445 0	12	217 38	Weld	317 6	1 52	0 85	4 90	1 88
5	7920	15 0	370 0	10	180 95	Weld	254 0	1 78	1 02	5 10	1 95
6	9504	18 0	370 0	10	180 95	Weld	211 7	2 12	1 23	5 12	2 31
7	11088	21 0	370 0	10	180 95	Weld	181 5	2 47	1 41	7 13	2 83
8	12672	24 0	370 0	10	180 95	Weld	158 8	2 80	1 81	8 15	2 93
9	14256	27 0	285 0	8	144 54	Weld	141 1	2 24	1 01	7 31	2 78
10	15840	30 0	285 0	8	144 54	Weld	127 0	2 47	1 12	8 13	3 01

#	Dist Top P1t FL (mm)	Dist Top Beam, DL (mm)	Beam End Mom FL 1 (kNm)	Beam End Mom FL 2 (kNm)	Hand1 MP	Hand2 MP	Adjusted wFL 1 (kN/m)	Adjusted wFL 2 (kN/m)
1	0 4	0 4	71 2	71 2	0 13	0 13	1128 5	1128 5
2	1 3	1 4	91 2	91 2	0 16	0 16	473 7	473 7
3	2 5	2 2	55 4	95 4	0 17	0 17	312 0	312 0
4	3 4	3 0	35 8	95 4	0 18	0 18	119 9	119 9
5	3 8	3 2	70 4	102 0	0 13	0 18	73 1	77 2
6	4 4	3 9	71 5	103 8	0 13	0 18	50 9	53 7
7	5 1	4 5	73 1	105 2	0 13	0 18	37 8	39 8
8	5 8	5 3	74 8	105 9	0 13	0 18	28 5	30 8
9	5 4	5 3	48 0	110 3	0 08	0 20	21 7	24 1
10	8 0	5 5	48 9	112 9	0 08	0 20	17 5	19 5

Table 9.4 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 85 % of the Ultimate Load (LSD Model)

(a) Output Page 1

W30X82 (Available in Canada and U.S.A.), Normally Used As A Beam	
SECTION PROPERTIES	
Nominal Mass,	M = 82 Kg/m
Cross Sectional Area,	A = 10600 mm ²
x-Axis Mom of Inertia,	I _x = 478.0x10 ⁶ mm ⁴
x-Axis Section Modulus,	S _x = 1810.0x10 ³ mm ³
x-Axis Plastic Modulus,	Z _x = 2070.0x10 ³ mm ³
y-Axis Mom of Inertia,	I _y = 20.3x10 ⁶ mm ⁴
y-Axis Section Modulus,	S _y = 184.0x10 ³ mm ³
y-Axis Plastic Modulus,	Z _y = 303.0x10 ³ mm ³
Torsional Constant,	J = 530.0x10 ³ mm ⁴
Warping Constant,	C _w = 1340.0x10 ⁶ mm ⁶
Overall Section Depth,	d = 528.0 mm
Flange Width,	b = 209.0 mm
Flange Thickness,	t = 13.3 mm
Clear Flange Distance,	ft = 85.5 mm
Web Height,	h = 501.4 mm
Web Thickness,	w = 9.5 mm
Clear Web Distance,	wt = 475.0 mm
MATERIAL PROPERTIES	
Yield Strength,	F _y = 300.0 MPa
Modulus of Elasticity,	E = 200000 MPa
SECTION CLASS & SECTION STRENGTH	
Flange Width/Thickness, b/t = 7.9,	CLASS 1
Web Width/Thickness, h/w = 52.8,	CLASS 1B
	CLASS 1V
Resistance Factor,	φ = 0.90
Fact. Yld. Moment Cap.,	M _y = 488.7 kNm
Fact. Moment Capacity,	M _r = 558.9 kNm
Fact. Shear Capacity,	V _r = 893.9 kN
Max. Shear Strength,	F _s = 188.0 MPa
	φ = 0.85F _y

FLEXIBLE END PLATE CONNECTION		FEP Fyp-DH-g-p-Bend-Dedg-Dtop-tp-L B Fub-Dd-# W Wu-F	
PLATE PROPERTIES		BOLT PROPERTIES	
Yield Strength,	F _{yp} = 300.0 MPa	Ultimate Strength,	F _{ub} = 825.0 MPa
Ultimate Strength,	F _{up} = 450.0 MPa	Resistance Factor,	φ _b = 0.85
Modulus of Elasticity,	E _p = 200000.0 MPa	Nominal Diameter,	D _b = 19.0 mm
Resistance Factor,	φ _p = 0.90	Fact. Tensile Cap.,	T _{rb} = 117.5 kN/Bolt
Bolt Hole Diameter,	D _h = 31.0 mm	Fact. Shear Capacity,	V _{rb} = 85.8 kN/Bolt
Bolt Hole Edge,	E = 140.0 mm	Bolt Cost,	COST _b = \$ 0.00/Bolt
Bolt Hole Pitch,	P = 75.0 mm	WELD PROPERTIES	
Bolt Hole End Dist.,	D _{end} = 35.0 mm	Ultimate Strength,	W _u = 480.0 MPa
Bolt Hole Edge Dist.,	D _{edg} = 30.0 mm	Resistance Factor,	φ _w = 0.85
Bolt Hole Top Dist.,	D _{top} = 75.0 mm	Pitlet Weld Leg Size,	L = 5.0 mm
Material Cost,	COST _p = \$ 1.00/kg	Fact. Long. Sh. Cap.,	V _{rw} = 0.75 kN/mm
		Pitlet Weld Cost,	COST _w = \$ 0.10/mm

UNIFORMLY DISTRIBUTED LOAD @ 85 % of the ULTIMATE LOAD LEVEL FOR THIS MEMBER.												
#	Beam Length (mm)	Length Depth (mm)	FACT LOAD (kN/m)	SPEC 100%DL (kN/m)	V _r V _f	M _r M _f	Max PL Defl (mm)	Max DL Defl (mm)	Length PLdefl	Length DLdefl	Bend PL (rads)	Bend DL (rads)
1	1584	3.0	859.3	787.4	1.2	1.9	0.8	0.7	1830	2412	0.0017	0.0013
2	3168	6.0	378.7	302.9	1.5	1.3	5.3	4.1	811	784	0.0052	0.0042
3	4752	9.0	188.3	134.5	2.2	1.2	11.7	9.3	407	805	0.0079	0.0063
4	6336	12.0	94.7	75.7	3.0	1.2	20.7	16.8	308	382	0.0105	0.0084
5	7920	15.0	60.8	48.5	3.7	1.2	32.4	25.9	244	356	0.0131	0.0106
6	9504	18.0	42.1	33.7	4.5	1.2	48.7	37.3	204	308	0.0157	0.0128
7	11088	21.0	30.9	24.7	5.2	1.2	63.8	50.4	175	218	0.0183	0.0147
8	12672	24.0	23.7	18.9	6.0	1.2	82.9	68.4	153	191	0.0208	0.0168
9	14256	27.0	18.7	15.0	6.7	1.2	105.0	84.0	138	170	0.0235	0.0189
10	15840	30.0	15.1	12.1	7.5	1.2	129.8	103.7	123	153	0.0263	0.0209

Table 9.4 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 85 % of the Ultimate Load (LSD Model)

(b) Output Page 2

W30x82 (Available In Canada And U.S.A.), Normally Used As A Beam											
CONN #1 : FEP(300-21-140- 75-35-30- 75- 8-L) S1 825-18-W) W(480- 5) LSD											
Support, Flange of a W30x75 Column, 18.8 mm thick. Material Prop. as per Supported Member. Supported member is uncoped.											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VF	Vweld VF	Vplt VF	Vbeam VF
1	1584	3.0	445.0	12	213.18	Weld	759.8	1.00	0.97	1.02	1.00
2	3168	6.0	445.0	12	213.18	Weld	599.8	1.27	1.23	1.30	1.25
3	4752	9.0	295.0	8	141.77	Weld	399.9	1.27	1.21	1.29	1.26
4	6336	12.0	220.0	6	108.07	Weld	299.9	1.27	1.24	1.28	1.27
5	7920	15.0	220.0	6	108.07	Weld	239.9	1.58	1.54	1.60	1.58
6	9504	18.0	295.0	8	141.77	RoTe	199.9	2.53	2.42	2.58	2.50
7	11088	21.0	370.0	10	177.48	RoTe	171.4	3.89	3.59	3.75	3.60
8	12672	24.0	370.0	10	177.48	RoTe	150.0	4.22	4.07	4.32	4.10
9	14256	27.0	370.0	10	177.48	RoTe	133.3	4.75	4.58	4.88	4.59
10	15840	30.0	445.0	12	213.18	RoTe	120.0	6.33	6.13	6.49	6.02
#	Dend Top Plt FL (mm)	Dend Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 MR	Mend2 MR	Adjusted wFL 1 (kN/m)	Adjusted wFL 2 (kN/m)			
1	0.8	0.5	21.9	21.9	0.04	0.04	1029.3	1029.3			
2	2.0	1.7	29.9	29.9	0.05	0.05	402.5	402.5			
3	2.0	1.8	13.8	31.7	0.02	0.06	173.2	179.5			
4	2.0	1.9	8.2	33.7	0.01	0.08	95.3	101.4			
5	2.5	2.4	8.4	35.3	0.01	0.08	81.7	85.1			
6	4.1	3.8	15.8	37.8	0.02	0.07	43.5	45.4			
7	8.0	5.8	28.3	39.8	0.05	0.07	32.6	33.8			
8	8.0	6.1	27.7	42.2	0.05	0.08	25.0	26.8			
9	7.7	6.8	28.1	44.5	0.05	0.08	19.8	20.5			
10	10.3	9.0	45.9	46.5	0.08	0.08	15.5	15.5			

CONN #2 : FEP(300-21-140- 75-35-30- 75- 8-L) S1 825-18-W) W(480- 5) LSD											
Support, Flange of a W30x75 Column, 18.8 mm thick. Material Prop. as per Supported Member. Supported member is uncoped.											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VF	Vweld VF	Vplt VF	Vbeam VF
1	1584	3.0	445.0	12	214.57	Weld	759.8	1.04	0.91	1.37	0.99
2	3168	6.0	445.0	12	214.57	Weld	599.8	1.32	1.16	1.73	1.21
3	4752	9.0	295.0	8	142.70	Weld	399.9	1.32	1.15	1.72	1.23
4	6336	12.0	220.0	6	106.78	Weld	299.9	1.32	1.14	1.71	1.24
5	7920	15.0	220.0	6	106.78	Weld	239.9	1.85	1.43	2.14	1.84
6	9504	18.0	220.0	6	106.78	RoTe	199.9	1.98	1.71	2.57	1.84
7	11088	21.0	295.0	8	142.70	RoTe	171.4	3.07	2.89	4.02	2.80
8	12672	24.0	295.0	8	142.70	RoTe	150.0	3.51	3.07	4.59	3.21
9	14256	27.0	370.0	10	178.84	RoTe	133.3	4.94	4.32	5.48	4.30
10	15840	30.0	370.0	10	178.84	RoTe	120.0	5.49	4.78	7.19	4.83
#	Dend Top Plt FL (mm)	Dend Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mend1 MR	Mend2 MR	Adjusted wFL 1 (kN/m)	Adjusted wFL 2 (kN/m)			
1	0.5	0.4	37.4	37.4	0.07	0.07	1078.8	1078.8			
2	1.8	1.5	49.0	49.0	0.09	0.09	417.7	417.7			
3	1.5	1.7	22.8	51.5	0.04	0.09	178.3	188.5			
4	1.8	1.8	13.5	53.1	0.02	0.10	87.3	105.3			
5	2.4	2.3	13.5	55.4	0.02	0.10	62.3	67.7			
6	2.8	2.7	13.9	58.8	0.02	0.10	43.3	47.1			
7	4.6	4.3	25.3	59.1	0.05	0.11	32.8	34.8			
8	8.2	4.7	25.8	61.4	0.05	0.11	24.9	28.7			
9	7.2	8.5	42.7	63.8	0.08	0.11	20.4	21.2			
10	8.1	7.2	43.8	66.3	0.08	0.12	15.5	17.3			

Table 9.4 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 85 % of the Ultimate Load (LSD Model)

(c) Output Page 3

W360x82 (Available in Canada And U.S.A.), Normally Used As A Beam											
CONN #3 : FEP(300-21-140- 75-35-30- 75-10-L) S(825-19-F) W(480- S) LSD											
Support, Flange of a W360x79 Column, 18 mm thick Material Prop. as per Supported Member Supported member is uncoped											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VV	Vweld VV	Vplt VV	Vbeam VV
1	1584	3.0	445.0	12	215.87	Weld	789.8	0.96	0.74	1.71	0.87
2	3168	8.0	445.0	12	215.87	Weld	899.8	1.22	0.94	2.15	1.15
3	4752	9.0	370.0	10	179.79	Weld	399.9	1.50	1.12	2.70	1.45
4	6336	12.0	295.0	8	143.82	Weld	399.9	1.80	1.31	2.87	1.54
5	7920	15.0	220.0	6	107.45	Weld	239.9	1.82	1.18	2.87	1.47
6	9504	18.0	220.0	6	107.45	Weld	199.9	1.82	1.39	3.31	1.78
7	11088	21.0	220.0	6	107.45	Weld	171.4	2.13	1.82	3.74	2.03
8	12672	24.0	295.0	8	143.82	Weld	150.0	3.20	2.42	5.74	2.94
9	14256	27.0	295.0	8	143.82	Weld	133.3	3.59	2.72	6.45	3.32
10	15840	30.0	370.0	10	179.79	Weld	120.0	4.88	3.89	9.99	4.90
#	Dist Top Flt. (mm)	Dist Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mand1 MP	Mand2 MP	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	0.4	0.4	53.0	53.0	0.09	0.09	1128.3	1128.3			
2	1.8	1.8	70.0	70.0	0.13	0.13	434.5	434.5			
3	2.1	1.9	50.0	72.8	0.09	0.13	185.0	194.0			
4	2.2	2.1	32.4	74.8	0.08	0.13	101.3	109.8			
5	2.1	2.1	19.3	77.4	0.03	0.14	83.1	70.5			
6	2.8	2.8	19.8	78.8	0.04	0.14	43.8	49.1			
7	3.1	3.0	30.1	81.1	0.04	0.15	32.2	38.2			
8	4.7	4.4	38.2	83.5	0.06	0.15	25.5	27.8			
9	5.4	5.0	38.5	85.7	0.07	0.15	20.1	22.1			
10	7.8	6.7	59.2	88.0	0.11	0.15	17.0	18.0			

CONN #4 : FEP(300-21-140- 75-35-30- 75-12-L) S(825-19-F) W(480- S) LSD											
Support, Flange of a W360x79 Column, 18 mm thick Material Prop. as per Supported Member Supported member is uncoped											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt VV	Vweld VV	Vplt VV	Vbeam VV
1	1584	3.0	445.0	12	217.38	Weld	789.8	0.77	0.49	2.05	0.94
2	3168	8.0	445.0	12	217.38	Weld	899.8	0.99	0.45	3.20	1.05
3	4752	9.0	445.0	12	217.38	Weld	399.9	1.29	0.58	3.59	1.58
4	6336	12.0	445.0	12	217.38	Weld	399.9	1.72	0.90	5.19	1.98
5	7920	15.0	370.0	10	180.95	Weld	239.9	1.88	1.08	5.40	2.10
6	9504	18.0	370.0	10	180.95	Weld	199.9	2.25	1.29	5.48	2.42
7	11088	21.0	370.0	10	180.95	Weld	171.4	2.82	1.50	7.55	2.81
8	12672	24.0	370.0	10	180.95	Weld	150.0	3.98	1.71	8.83	3.14
9	14256	27.0	295.0	8	144.54	Weld	133.3	3.39	1.08	7.74	3.89
10	15840	30.0	295.0	8	144.54	Weld	120.0	2.92	1.19	6.50	3.23
#	Dist Top Flt. FL (mm)	Dist Top Beam, DL (mm)	Beam End Mom. FL 1 (kNm)	Beam End Mom. FL 2 (kNm)	Mand1 MP	Mand2 MP	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	0.4	0.3	59.8	59.8	0.12	0.12	1128.8	1128.8			
2	1.4	1.3	80.5	80.5	0.19	0.19	450.8	450.8			
3	2.2	3.0	95.7	85.7	0.17	0.17	202.3	202.3			
4	3.1	2.8	95.8	95.8	0.19	0.18	114.4	114.4			
5	3.3	3.0	99.8	100.8	0.13	0.18	89.8	73.4			
6	4.0	3.7	71.8	102.6	0.13	0.18	48.4	51.1			
7	4.7	4.3	72.3	105.0	0.13	0.18	35.6	37.7			
8	5.4	5.0	73.9	107.4	0.13	0.19	27.3	29.0			
9	4.8	4.8	48.0	108.8	0.09	0.20	20.8	23.0			
10	5.8	5.2	48.2	111.0	0.09	0.20	18.7	18.7			

Table 9.5 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 95 % of the Ultimate Load (SVS Model)

(a) Output Page 1

W310X32		(Available in Canada And U.S.A.). Normally Used As A Beam.	
SECTION PROPERTIES		MATERIAL PROPERTIES	
Nominal Mass,	M = 82 KG/M	Yield Strength,	F _y = 300.0 MPa
Cross Sectional Area,	A = 10500 mm ²	Modulus of Elasticity,	E = 200000 MPa
x-Axis Mom. of Inertia,	I _x = 479 0x10 ⁸ mm ⁴	SECTION CLASS & SECTION STRENGTH	
x-Axis Section Modulus,	S _x = 1810 0x10 ³ mm ³	Flange Width/Thickness, D/t = 7.5,	CLASS 1
x-Axis Plastic Modulus,	Z _x = 2070 0x10 ³ mm ³	Web Width/Thickness, h/t _w = 52.8,	CLASS 1B
y-Axis Mom. of Inertia,	I _y = 30 3x10 ⁸ mm ⁴		CLASS 1V
y-Axis Section Modulus,	S _y = 194 0x10 ³ mm ³	Resistance Factor,	φ = 0.90
y-Axis Plastic Modulus,	Z _y = 303 0x10 ³ mm ³	Fact. Yld. Moment Cap.,	M _y = 488.7 kNm
Torsional Constant,	J = 830 0x10 ³ mm ⁴	Fact. Moment Capacity,	M _r = 555.9 kNm
Warping Constant,	C _w = 1380 0x10 ⁸ mm ⁶	Fact. Shear Capacity,	V _r = 583.9 kN
Overall Section Depth,	D = 325.0 mm	Max. Shear Strength,	F _s = 198.0 MPa
Flange Width,	b = 305.0 mm		φ = 0.85 F _y
Flange Thickness,	t = 13.3 mm		
Clear Flange Distance,	FT = 85.8 mm		
Web Height,	h = 501.4 mm		
Web Thickness,	w = 9.5 mm		
Clear Web Distance,	WT = 470.0 mm		

FLEXIBLE END PLATE CONNECTION		PEP(Fyb-Dh-g-p-Band-Bedg-Dtop-tp-L) B(Fud-Dh-p) W(LU-F)	
PLATE PROPERTIES		BOLT PROPERTIES	
Yield Strength,	F _{yp} = 300.0 MPa	Ultimate Strength,	F _{ud} = 525.0 MPa
Ultimate Strength,	F _{up} = 450.0 MPa	Resistance Factor,	φ _b = 0.87
Modulus of Elasticity,	E _p = 200000.0 MPa	Nominal Diameter,	D _b = 19.0 mm
Resistance Factor,	φ _p = 0.90	Fact. Tensile Cap.,	T _{pb} = 117.8 kN/Bolt
Bolt Hole Diameter,	D _h = 21.0 mm	Fact. Shear Capacity,	V _{pb} = 85.5 kN/Bolt
Bolt Hole Gage,	G = 150.0 mm	Bolt Cost,	COST _B = \$ 10.00/Bolt
Bolt Hole Pitch,	P = 75.0 mm	WELD PROPERTIES	
Bolt Hole End Dist.,	D _{end} = 35.0 mm	Ultimate Strength,	F _u = 480.0 MPa
Bolt Hole Edge Dist.,	D _{edg} = 30.0 mm	Resistance Factor,	φ _w = 0.87
Bolt Hole Top Dist.,	D _{top} = 75.0 mm	Fillet Weld Leg Size,	a = 9.0 mm
Material Cost,	COST _P = \$ 1.00/KG	Fact. Long. Sh. Cap.,	V _{pw} = 0.75 kN/mm
		Fillet Weld Cost,	COST _W = \$ 0.10/mm

UNIFORMLY DISTRIBUTED LOAD @ 95 % OF THE ULTIMATE LOAD LEVEL FOR THIS MEMBER												
#	Beam Length (mm)	Length Depth	FACT LOAD (kN/m)	SPEC 100%DL (kN/m)	V _r V _f	M _r M _f	Max FL Defl (mm)	Max DL Defl (mm)	Length Floorfl	Length DLoeffl	End FL (kN)	End DL (kN)
1	1584	3.0	1072.3	887.7	1.1	1.7	0.9	0.7	1737	2159	0.0019	0.0016
2	3168	8.0	423.2	338.6	1.3	1.1	5.6	4.6	488	583	0.0085	0.0047
3	4752	8.0	188.1	150.5	2.0	1.1	18.5	10.4	308	458	0.0101	0.0070
4	6336	12.0	105.8	84.9	2.7	1.1	28.3	18.6	234	342	0.0136	0.0094
5	7920	15.0	87.7	64.2	3.3	1.1	44.9	29.0	178	273	0.0171	0.0117
6	9504	19.0	67.0	37.5	4.0	1.1	65.2	41.7	146	228	0.0208	0.0140
7	11088	21.0	34.5	27.8	4.7	1.1	85.8	58.8	130	195	0.0238	0.0166
8	12672	24.0	28.5	21.2	5.3	1.1	113.1	74.2	112	171	0.0272	0.0187
9	14256	27.0	20.9	16.7	6.0	1.1	144.3	93.9	88	152	0.0307	0.0211
10	15840	30.0	15.9	13.5	6.7	1.1	179.7	115.8	68	137	0.0342	0.0234

Table 9.5 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 95 % of the Ultimate Load (SVS Model)

(b) Output Page 2

W30x82 (Available in Canada And U.S.A.), Normally Used As A Beam											
CONN #1 FEP(300-21-140- 75-35-30- 75- 8-L) S(825-19-F) W(480- 5) SVS											
Support: Flange of a W30x79 Column, 18.8 mm thick Material Prop. as per Supported Member Supported member is uncoped											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt Vt	Vweld Vt	Valt Vt	Vbeam Vt
1	1584	3 0	445 0	12	213.18	Weld	849.2	0.82	0.80	0.92	0.89
2	3168	6 0	445 0	12	213.18	Weld	870.4	1.18	1.01	1.18	1.12
3	4752	9 0	295 0	8	141.77	Weld	448.9	1.18	1.01	1.18	1.12
4	6336	12 0	295 0	8	141.77	Weld	325.2	1.57	1.34	1.54	1.48
5	7920	15 0	230 0	6	106.07	Weld	288.2	1.47	1.25	1.44	1.39
6	9504	18 0	230 0	6	106.07	Weld	223.5	1.77	1.60	1.72	1.67
7	11088	21 0	145 0	4	70.38	Weld	181.5	1.37	1.15	1.32	1.28
8	12672	24 0	145 0	4	70.38	Weld	167.6	1.57	1.32	1.51	1.46
9	14256	27 0	145 0	4	70.38	Weld	149.0	1.77	1.48	1.70	1.65
10	15840	30 0	145 0	4	70.38	Weld	134.1	1.98	1.65	1.89	1.83
#	Dist Top Pts FL (mm)	Dist Top Beam, DL (mm)	Beam End Mom FL 1 (kNm)	Beam End Mom FL 2 (kNm)	Moment Mr	Moment Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	-	-	-	-	-	-	-	-			
2	-	-	-	-	-	-	-	-			
3	-	-	-	-	-	-	-	-			
4	-	-	-	-	-	-	-	-			
5	-	-	-	-	-	-	-	-			
6	-	-	-	-	-	-	-	-			
7	-	-	-	-	-	-	-	-			
8	-	-	-	-	-	-	-	-			
9	-	-	-	-	-	-	-	-			
10	-	-	-	-	-	-	-	-			

CONN #2 FEP(300-21-140- 75-35-30- 75- 8-L) S(825-19-F) W(480- 5) SVS											
Support: Flange of a W30x79 Column, 18.8 mm thick Material Prop. as per Supported Member Supported member is uncoped											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn. Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt Vt	Vweld Vt	Valt Vt	Vbeam Vt
1	1584	3 0	445 0	12	214.57	Weld	849.2	0.83	0.80	1.22	0.89
2	3168	6 0	445 0	12	214.57	Weld	870.4	1.18	1.01	1.65	1.12
3	4752	9 0	295 0	8	142.70	Weld	448.9	1.18	1.01	1.64	1.12
4	6336	12 0	295 0	8	142.70	Weld	325.2	1.57	1.34	2.08	1.48
5	7920	15 0	230 0	6	106.78	Weld	288.2	1.47	1.25	1.81	1.39
6	9504	18 0	230 0	6	106.78	Weld	223.5	1.77	1.60	2.30	1.67
7	11088	21 0	145 0	4	70.82	Weld	181.5	1.37	1.15	1.77	1.28
8	12672	24 0	145 0	4	70.82	Weld	167.6	1.57	1.32	2.02	1.46
9	14256	27 0	145 0	4	70.82	Weld	149.0	1.77	1.48	2.27	1.65
10	15840	30 0	145 0	4	70.82	Weld	134.1	1.98	1.65	2.52	1.83
#	Dist Top Pts FL (mm)	Dist Top Beam, DL (mm)	Beam End Mom FL 1 (kNm)	Beam End Mom FL 2 (kNm)	Moment Mr	Moment Mr	Adjusted wPL 1 (kN/m)	Adjusted wPL 2 (kN/m)			
1	-	-	-	-	-	-	-	-			
2	-	-	-	-	-	-	-	-			
3	-	-	-	-	-	-	-	-			
4	-	-	-	-	-	-	-	-			
5	-	-	-	-	-	-	-	-			
6	-	-	-	-	-	-	-	-			
7	-	-	-	-	-	-	-	-			
8	-	-	-	-	-	-	-	-			
9	-	-	-	-	-	-	-	-			
10	-	-	-	-	-	-	-	-			

Table 9.5 Flexible End Plate Connection Design for a Simply Supported, Continuously Braced, Standard Steel Beam, with a Uniformly Distributed Load of 95 % of the Ultimate Load (SVS Model)

(c) Output Page 3

W30x132 (Available in Canada And U.S.A.). Normally Used As A Beam											
CONN #3 : FEP(300-21-140- 75-35-30- 75-10-L) S: 825-18-W: W1480- S: SVS											
Support: Flange of a W30x132 Column: 18 mm thick Material Prop: 45 per Supported Member Supported member is uncoped											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vlt V _f	Vbeam V _f
1	1584	3 0	445 0	12	215 97	Weld	849 2	0 93	0 80	1 53	0 89
2	3168	6 0	445 0	12	215 97	Weld	870 4	1 18	1 01	1 94	1 12
3	4752	9 0	295 0	8	143 82	Weld	445 9	1 18	1 01	1 92	1 12
4	6336	12 0	295 0	8	107 45	Weld	335 2	1 57	1 34	2 57	1 49
5	7920	15 0	230 0	6	107 45	Weld	288 2	1 47	1 25	2 39	1 39
6	9504	18 0	230 0	6	71 37	Weld	223 5	1 77	1 50	2 87	1 57
7	11088	21 0	145 0	4	71 37	Weld	181 5	1 37	1 15	2 21	1 28
8	12672	24 0	145 0	4	71 37	Weld	157 5	1 37	1 32	2 52	1 45
9	14256	27 0	145 0	4	71 37	Weld	145 0	1 77	1 48	3 44	1 65
10	15840	30 0	145 0	4	71 37	Weld	134 1	1 98	1 85	3 78	1 83
#	Dist Top Fl (mm)	Dist Top Beam, Sl (mm)	Beam End Mom FL 1 (kNm)	Beam End Mom FL 2 (kNm)	Mand1 MP	Mand2 MP	Adjusted wFL 1 (kN/m)	Adjusted wFL 2 (kN/m)			
1	-	-	-	-	-	-	-	-			
2	-	-	-	-	-	-	-	-			
3	-	-	-	-	-	-	-	-			
4	-	-	-	-	-	-	-	-			
5	-	-	-	-	-	-	-	-			
6	-	-	-	-	-	-	-	-			
7	-	-	-	-	-	-	-	-			
8	-	-	-	-	-	-	-	-			
9	-	-	-	-	-	-	-	-			
10	-	-	-	-	-	-	-	-			

CONN #4 : FEP(300-21-140- 75-35-30- 75-12-L) S: 825-18-W: W1480- S: SVS											
Support: Flange of a W30x132 Column: 18 mm thick Material Prop: 45 per Supported Member Supported member is uncoped											
#	Beam Length (mm)	Length Depth (mm)	Plate Length (mm)	No. of Bolts	Conn Cost (\$)	Governing Condition	FACT SHEAR (kN)	Vbolt V _f	Vweld V _f	Vlt V _f	Vbeam V _f
1	1584	3 0	445 0	12	217 35	Weld	849 2	0 93	0 80	1 53	0 89
2	3168	6 0	445 0	12	217 35	Weld	870 4	1 18	1 01	2 32	1 12
3	4752	9 0	295 0	8	144 84	Weld	445 9	1 18	1 01	2 31	1 12
4	6336	12 0	295 0	8	108 94	Weld	335 2	1 57	1 34	3 08	1 49
5	7920	15 0	230 0	6	108 13	Weld	288 2	1 47	1 25	2 87	1 39
6	9504	18 0	230 0	6	71 73	Weld	223 5	1 77	1 50	3 44	1 57
7	11088	21 0	145 0	4	71 73	Weld	181 5	1 37	1 15	2 85	1 28
8	12672	24 0	145 0	4	71 73	Weld	157 5	1 37	1 32	3 03	1 45
9	14256	27 0	145 0	4	71 73	Weld	145 0	1 77	1 48	3 41	1 65
10	15840	30 0	145 0	4	71 73	Weld	134 1	1 98	1 85	3 78	1 83
#	Dist Top Fl (mm)	Dist Top Beam, Sl (mm)	Beam End Mom FL 1 (kNm)	Beam End Mom FL 2 (kNm)	Mand1 MP	Mand2 MP	Adjusted wFL 1 (kN/m)	Adjusted wFL 2 (kN/m)			
1	-	-	-	-	-	-	-	-			
2	-	-	-	-	-	-	-	-			
3	-	-	-	-	-	-	-	-			
4	-	-	-	-	-	-	-	-			
5	-	-	-	-	-	-	-	-			
6	-	-	-	-	-	-	-	-			
7	-	-	-	-	-	-	-	-			
8	-	-	-	-	-	-	-	-			
9	-	-	-	-	-	-	-	-			
10	-	-	-	-	-	-	-	-			

10. Summary and Conclusions

10.1 Summary and Conclusions

1. A two part study has been made in which a Limit States Design model for flexible end plate connections has been developed and then used to design standardized connections.

PART A

2. A review of literature on experimental research indicated that flexible end plate connections were designed to transfer shear only. This was based on the premise that relatively small moments were developed and that the rotational characteristics were independent of the shear-to-moment ratio. However, no assessment of the ultimate shear strength of the connections was made.

3. The connection design handbooks from five different countries indicate that it is the current general practice to design flexible end plate connections to transfer primary shear only and to disregard the secondary forces which develop in the connection when it rotates. However, the AustISC (1978) handbook includes some empirical rules which are intended to compensate for the secondary forces. Empirical rules are also used to ensure adequate flexibility of such connections. Evaluation of the maximum unrestrained rotation of the connection is attempted only in the

Australian (AustISC 1978) and British (BCSA 1982) handbooks.

However, the method used is known to be unconservative.

4. A complete design model, in order to ensure a consistent margin of safety, should evaluate

- i. the primary shear strength of the connection in the presence of secondary forces,
- ii. the end moment developed at the factored load level, and
- iii. the maximum unrestrained rotation of the connection at which bottom flange bearing occurs.

5. With modifications to the load-deformation relationships originally given by Hafez (1982) and by Kennedy and Hafez (1984), the analytical method developed by Hafez (1982) has been used to predict the moment-rotation relationship of 28 full-scale connections. The predictions correlate well with the test results obtained prior to the occurrence of bottom flange bearing of the supported member on the support has been obtained.

6. The analytical prediction of the moment immediately prior to bottom flange bearing is very good, with a mean value of the test-to-predicted ratio of the 28 tests being 1.06 and the coefficient of variation being 0.103. However, the corresponding test-to-predicted ratio for connection rotation and coefficient of variation when bottom flange bearing is imminent are not as good for these tests, and were calculated to be, respectively, 1.08 and 0.253.

7. While the analytical method was developed for uncoped members with stiff supports, it will likely result in accurate or conservative overestimates of the connection moment at a given rotation when applied to other types of connection details.

8. When the support is not rigid, as is assumed in the analytical method developed, it is possible that bottom flange bearing could occur at smaller rotations than those predicted by the analysis.

9. To prevent the undesirable situation of premature bottom flange bearing from occurring, due to conditions discussed above in items 6 and 8, it is suggested that the unrestrained rotation limit obtained using the analysis be reduced by a factor of $2/3$ for design purposes.

10. An analytical model based on experimental studies reported in the literature was developed to predict the magnitude and distribution of secondary forces in the bolts. The model also allows the prediction of end plate and bolt rupture due to excessive connection deformation. The Hafez (1982) analytical method, with modifications, is used to predict the secondary forces in the supported member web, the fillet welds, and the end plate.

11. By applying lower bound plasticity theories, a series of resistance equations were developed for the prediction of the primary shear resistance of each of the connection components (the bolts, the end plate, the welds, and the supported beam web), taking into account the

secondary forces acting in the connection. In some cases, the resulting resistance equations are less conservative (but possibly more rational) than the current guidelines set out in the governing design standard, CAN3-S16.1-M78 (CSA 1978).

12. The flexible end plate connection, depending upon its size relative to the depth of the supported member and upon whether or not the member end is coped, may or may not provide significant torsional restraint. It is for this reason that the three-part design model proposed herein, which does not consider this point, is recommended for use only when the supported member has sufficient bracing along its length so that this condition is of no concern.

PART B

13. Existing connection design handbooks limit users to a limited number of geometric and material properties. One alternative is for fabricators to develop their own 'shop standards'.

14. Computer programs can be developed to allow the rapid design of standard connection details possessing any combination of geometric and material properties provided that the connection design models are general enough in nature to encompass a wide range of applications.

15. An interactive computer program for the design of standardized flexible end plate connections, based on the design model developed herein, is developed to provide one

possible software solution.

16. The computer program designs connections at the factored load level. In some instances this may cause inelastic response of the supported members. Ignoring this condition can result in inadequate connection details at the factored load level.

17. The computer program is useful to both fabricators and consulting engineers and can be applied to the solution of several different design problems.

10.2 Areas of Further Research

Areas for suggested further study are:

PART A

1. An experimental program to verify the connection resistance equations proposed in this thesis as there have been no experimental studies conducted specifically for the evaluation of the ultimate shear capacity of flexible end plates. This would require that a series of connection details be designed to fail in the bolts, in the plate, in the welds, and in the supported member web at factored load levels that could be attained in standard steel building frames.

2. An experimental and analytical investigation into the behaviour of skewed flexible end plate connection details. Such a study would determine whether or not the

design model proposed herein is generally applicable or not and would identify any unique behavioural characteristics of skewed connections at factored load levels.

3. An experimental program to verify or improve on the load-deformation models developed in Chapter 5 for use in determining the magnitude and distribution of secondary forces in the connection.

4. An experimental and analytical investigation into the behaviour of flexible end plate connections subjected to dynamic loading or load reversals. Such a study could evaluate the value of using this type of connection detail in seismic regions.

5. An analytical or statistical study investigating the minimum shear force for which connection details should be designed. It is not clear whether a minimum size connection detail should be specified, when the factored shear force is relatively small, in order to account for torsional and other effects at the connection not normally considered.

PART B

6. Expand the CDSSSS program capabilities so that individual connection details for specific span lengths and loading conditions can be designed; the current approach gives connection designs for a wide range of beam spans and loading conditions but does not deal with any specific

situations.

7. Adjust the CDSSSS program so that connection details framing back-to-back into a common support can be considered. This would require an evaluation of the geometric considerations and of the strength of such a detail. ✓

8. Add a graphics capability to the CDSSSS program so that connection details could be designed and detailed in one step.

9. Incorporate the connection design model from PART A into a frame design program, along with other connection design models, so that beam-girder framing systems could be designed and connections detailed simultaneously.

10. With the development of a method for predicting factored level connection rotations for composite beams, incorporate the design model developed in PART A, or a modification of it, into the CDSSSS program for use in connection design for composite construction.

11. Modify the CDSSSS program so it can design connections for built-up members and plate girders.

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Appendix A

Computer Program for the Analysis of the Moment in a
Flexible End Plate Connection

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2

Card # 1,

TITLE = a title of up to 40 characters in length
can be used to describe the specimen

Card # 2,

G = bolt hole gage
P = bolt hole pitch for full size connection
specimens and the plate width for tension
specimens
E = the edge distance of end plate measured from
the center line of bolt hole to nearest edge
of plate
T = end plate thickness
F = leg size of fillet weld joining the end
plate to supported beam web
W = thickness of the supported beam web
DEPTH P = the total depth of the end plate
DEPTH B = the total depth of the supported member
TOPDIS = the distance from the top of the supported
member to the top of the end plate
DH = bolt hole diameter
DB = nominal bolt diameter

Card # 3,

FYP = tensile yield strength of the end plate
FUP = tensile ultimate strength of the end plate
EP = Young's Modulus of elasticity of the end
plate
FUB = tensile ultimate strength of the bolt
FY = tensile yield strength of the supported
beam
FU = tensile ultimate strength of the supported
beam

UNITS = 1 for IMPERIAL, 2 for SI (when IMPERIAL
units are used all dimensions must be in
inches and all material strengths in ksi,
for SI units, dimensions must be in mm
and material strengths in MPa)

TABLE = insert the table number or identification,
up to 4 characters in each of the two
variables, an example table heading is:

TABLE 5.12

to obtain this you would input 5.12 in
variable TABLE(1) and leave variable
TABLE(2) blank.

NTYPE = set equal to 1 if tabular output is desired

```

C          and set equal to 2 if plot compatible
C          output is desired.
C
C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
      REAL MOMEN1,MOMEN2,MOMEN3,MUP,NULT
C
      DIMENSION CARM1(200),CARM2(200),CARM3(200),CF1(200),
&              CF2(200),CF3(200),DELTC1(200),DELTC2(200),
&              DELTC3(200),DELTT1(200),DELTT2(200),
&              DELTT3(200),ENDM1(20),ENDM2(20),ENDM3(20),
&              TABLE(5),TITLE(20),TARM1(200),TARM2(200),
&              TARM3(200),TEN1(200),TEN2(200),TEN3(200)
C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
C*****READ STATEMENTS:
C
C
      1 CONTINUE
      READ(5,10)TITLE(1),TITLE(2),TITLE(3),TITLE(4),
&              TITLE(5),TITLE(6),TITLE(7),TITLE(8),
&              TITLE(9),TITLE(10)
      10 FORMAT(10A4)
      READ(5,20)G,P,E,T,F,W,DEPTHHP,DEPTHB,TOPDIS,DH,DB,
&              FYP,FUP,EP,FUB,FY,FU,UNITS,TABLE(1),
&              TABLE(2),NTYPE
      20 FORMAT(11F7.2,/,7F9.2,2A4,11)
      IF(G.LE.0)STOP
C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
C*****CONVERSION OF SI DIMENSIONS TO IMPERIAL:
C
C
      IF(UNITS.GE.2)CALL CONV1(DB,DEPTHB,DEPTHHP,DH,E,EP,
&              F,FY,FYP,FU,FUB,FUP,G,P,T,TOPDIS,W )
C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
C*****CALCULATION OF CONSTANTS:
C
C
      PI = 3.1416
      CLEARL = (G - W - DH - 2*F)/2.
C
      DELTAE = (CLEARL**2*FUP)/(2.*EP*T)
      ALPHAE = ATAN2(DELTAE,CLEARL)
C
      VULT = (0.60*0.70*PI*(DB**2/4)*FUB)/P
      HMAX = VULT
      NULT = (P - DH)*T*FUP/P

```

```

      IF(NULT.LT.HMAX)HMAX = NULT
      BULT = E*T*FUP/P
      IF(E.GT.(3.*DB))BULT = 3.*DB*T*FUP/P
      IF(BULT.LT.HMAX)HMAX = BULT
C
      MUP = 0.25*(T**2)*FUP
      PYP = T*FYP
C
      DELTAP = 4.*HMAX*MUP/(PYP**2)
      ALPHAP = ATAN2(DELTAP,CLEARL)
C
      BOTTOM = DEPTHB - DEPTH - TOPDIS
C
      CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
C*****CALCULATION OF LIMITING ROTATION & MOMENT OF
C      CONNECTION:
C
C
C      PART = DEPTH/100
      I = 0
2050 I = I + 1
C
      IF(I.GE.100)CODE = 10
      IF(I.GE.100)CALL ERROR(CODE)
C
      TLENGT = PART*(100 - I)
      CLENGT = PART*I
      ADJACE = CLENGT + BOTTOM
      THETA = ATAN2(T,ADJACE)
C
      IF(I.GT.1.AND.CFORC1.GT.TFORC1)GO TO 2010
      IMAX1 = I
      THETA1 = THETA
      CALL COMP1(CARM1,CF1,CFORC1,CLENGT,DELTC1,FU,I,
&              PART,THETA,W)
C
      CALL TENS1(CLEARL,DELTA,DELTAP,DELTT1,EP,HMAX,
&              I,MUP,PYP,PART,T,TARM1,TEN1,TFORC1,THETA)
C
2010 IF(I.GT.1.AND.CFORC2.GT.TFORC2)GO TO 2020
      IMAX2 = I
      THETA2 = THETA
      CALL COMP2(CARM2,CF2,CFORC2,CLENGT,DELTC2,FU,I,
&              PART,THETA,W)
      CALL TENS2(CLEARL,DELTA,DELTAP,DELTT2,DH,EP,HMAX,
&              I,MUP,P,PYP,PART,T,TARM2,TEN2,TFORC2,THETA)
C
2020 IF(I.GT.1.AND.CFORC3.GT.TFORC3)GO TO 2030
      IMAX3 = I
      THETA3 = THETA
      CALL COMP2(CARM3,CF3,CFORC3,CLENGT,DELTC3,FU,I,
&              PART,THETA,W)
C

```



```

      CALL TENS3(CLEARL,DELTAE,DELTAP,DELTT3,EP,HMAX,I,MUP,
&              PYP,PART,T,TARM3,TEN3,TFORC3,THETA )
C
      GO TO 2050
C
2030 IF(CFORC1.GT.TFORC1.AND.CFORC2.GT.TFORC2.AND.
&CFORC3.GT.TFORC3)GO TO 2060
      GO TO 2050
C
2060 CALL MOMENT(CARM1,CF1,IMAX1,PART,TARM1,TEN1,TOTALM)
      MOMEN1 = TOTALM
C
      CALL MOMENT(CARM2,CF2,IMAX2,PART,TARM2,TEN2,TOTALM)
      MOMEN2 = TOTALM
C
      CALL MOMENT(CARM3,CF3,IMAX3,PART,TARM3,TEN3,TOTALM)
      MOMEN3 = TOTALM
C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
C*****CALCULATION OF MOMENT ROTATION RELATIONSHIP:
C
C
      DO 3000 N = 1,10
      IF(N.LE.5)THETA = 0.001*N
      IF(N.GT.5)THETA = 0.005 + (N - 5)*0.005
C
      THETA = 0.0025*N
      I = 0
3050 I = I + 1
C
      IF(I.GE.100)CODE = 20
      IF(I.GE.100)CALL ERROR(CODE)
C
      TLENGT = PART*(100 - I)
      CLENGT = PART*I
C
      IF(THETA.GT.THETA1)STOP1 = N
      IF(THETA.GT.THETA1)GO TO 3010
C
      IF(I.GT.1.AND.CFORC1.GT.TFORC1)GO TO 3010
      IMAX1 = I
      CALL COMP1(CARM1,CF1,CFORC1,CLENGT,DELTC1,FU,I,
&              PART,THETA,W)
      CALL TENS1(CLEARL,DELTAE,DELTAP,DELTT1,EP,HMAX,
&              I,MUP,PYP,PART,T,TARM1,TEN1,TFORC1,THETA)
C
3010 IF(THETA.GT.THETA2)STOP2 = N
      IF(THETA.GT.THETA2)GO TO 3020
      IF(I.GT.1.AND.CFORC2.GT.TFORC2)GO TO 3020
      IMAX2 = I
      CALL COMP2(CARM2,CF2,CFORC2,CLENGT,DELTC2,FU,
&              I,PART,THETA,W)
      CALL TENS2(CLEARL,DELTAE,DELTAP,DELTT2,DH,EP,
&              HMAX,I,MUP,P,PYP,PART,T,TARM2,TEN2,TFORC2,THETA)

```



```

C
3020 IF(THETA.GT.THETA3)STOP3 = N
      IF(THETA.GT.THETA3)GO TO 3030
      IF(I.GT.1.AND.CFORC3.GT.TFORC3)GO TO 3030
      IMAX3 = I
      CALL COMP2(CARM3,CF3,CFORC3,CLENGT,DELTC3,FU,
&               I,PART,THETA,W)
C
      CALL TENS3(CLEARL,DELTAE,DELTAP,DELTT3,EP,HMAX,
&               I,MUP,PYP,PART,T,TARM3,TEN3,TFORC3,THETA)
C
      GO TO 3050
C
3030 IF(THETA.GT.THETA1.OR.CFORC1.GT.TFORC1)GO TO 3061
      GO TO 3050
3061 IF(THETA.GT.THETA2.OR.CFORC2.GT.TFORC2)GO TO 3062
      GO TO 3050
3062 IF(THETA.GT.THETA3.OR.CFORC3.GT.TFORC3)GO TO 3060
      GO TO 3050
C
3060 IF(THETA.GT.THETA1)ENDM1(N) = 0.0
      IF(THETA.GT.THETA1)GO TO 3070
      CALL MOMENT(CARM1,CF1,IMAX1,PART,TARM1,TEN1,TOTALM)
      ENDM1(N) = TOTALM
C
3070 IF(THETA.GT.THETA2)ENDM2(N) = 0.0
      IF(THETA.GT.THETA2)GO TO 3080
      CALL MOMENT(CARM2,CF2,IMAX2,PART,TARM2,TEN2,TOTALM)
      ENDM2(N) = TOTALM
C
3080 IF(THETA.GT.THETA3)ENDM3(N) = 0.0
      IF(THETA.GT.THETA3)GO TO 3090
      CALL MOMENT(CARM3,CF3,IMAX3,PART,TARM3,TEN3,TOTALM)
      ENDM3(N) = TOTALM
C
3090 CONTINUE
C
3000 CONTINUE
C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
C*****PRESENTATION OF OUTPUT:
C
C
      IF(UNITS.GE.2)
&CALL CONV2(DB,DEPTHB,DEPTHHP,DH,E,ENDM1,ENDM2,
&           ENDM3,EP,F,FY,FYP,FU,FUB,FUP,G,MOMEN1,
&           MOMEN2,MOMEN3,P,T,TPDIS,W )
C
      IF(NTYPE.EQ.2)GO TO 13
      CALL WRITE(DB,DEPTHB,DEPTHHP,DH,E,ENDM1,ENDM2,ENDM3,
&              EP,F,FY,FYP,FU,FUB,FUP,G,MOMEN1,MOMEN2,MOMEN3,
&              P,T,TABLE,THETA1,THETA2,THETA3,TITLE,TPDIS,
&              W)

```

```

      GO TO 14
13  CONTINUE
      CALL WRITE2(DB,DEPTHB,DEPTHHP,DH,E,ENDM1,ENDM2,ENDM3,
&      EP,F,FY,FYP,FU,FUB,FUP,G,MOMEN1,MOMEN2,MOMEN3,
&      P,T,TABLE,THETA1,THETA2,THETA3,TITLE,TOPDIS,
&      W)

```

```

14  CONTINUE

```

```

C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C

```

```

C*****RETURN TO EVALUATE ANOTHER END PLATE OR TO STOP
C      EXECUTION
C
C

```

```

      GO TO 1
      END

```

```

C
C*****
C*****
C

```

SUBROUTINE LISTINGS

```

C*****
C*****
C

```

```

      SUBROUTINE CONV1:

```

```

      Used to convert SI data to IMPERIAL data.

```

```

C
      SUBROUTINE CONV1(DB,DEPTHB,DEPTHHP,DH,E,EP,F,FY,FYP,
&      FU,FUB,FUP,G,P,T, TOPDIS,W )

```

```

C
      DB      = DB/25.4
      DEPTHB  = DEPTHB/25.4
      DEPTHHP = DEPTHHP/25.4
      DH      = DH/25.4
      E       = E/25.4
      EP      = EP/6.895
      F       = F/25.4
      FY      = FY/6.895
      FYP     = FYP/6.895
      FU      = FU/6.895
      FUB     = FUB/6.895
      FUP     = FUP/6.895
      G       = G/25.4
      P       = P/25.4
      T       = T/25.4
      TOPDIS  = TOPDIS/25.4
      W       = W/25.4

```

```

C
      RETURN
      END
C

```

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
 CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC

C
C
C
C
C
C

SUBROUTINE CONV2:

Used to convert IMPERIAL data to SI data.

SUBROUTINE CONV2(DB,DEPTHB,DEPTHHP,DH,E,ENDM1,
 & ENDM2,ENDM3,EP,F,FY,FYP,FU,FUB,
 & FUP,G,MOMEN1,MOMEN2,MOMEN3,P,T,TPDIS,W)

C
C
C
C

REAL MOMEN1,MOMEN2,MOMEN3

DIMENSION ENDM1(20),ENDM2(20),ENDM3(20)

DB = DB*25.4
 DEPTHB = DEPTHB*25.4
 DEPTHHP = DEPTHHP*25.4
 DH = DH*25.4
 E = E*25.4
 EP = EP*6.895
 F = F*25.4
 FY = FY*6.895
 FYP = FYP*6.895
 FU = FU*6.895
 FUB = FUB*6.895
 FUP = FUP*6.895
 G = G*25.4
 P = P*25.4
 T = T*25.4
 TOPDIS = TOPDIS*25.4
 W = W*25.4

C
C
C

MOMEN1 = MOMEN1*0.1130
 MOMEN2 = MOMEN2*0.1130
 MOMEN3 = MOMEN3*0.1130

DO 1000 N = 1,10
 ENDM1(N) = ENDM1(N)*0.1130
 ENDM2(N) = ENDM2(N)*0.1130
 ENDM3(N) = ENDM3(N)*0.1130

1000 CONTINUE

C
C

RETURN
 END

C
C
C

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
 CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC

C
C
C
C

SUBROUTINE COMP1:

Calculates the compressive force distribution in a
 flexible end plate connection using the PROPOSED

```

compression model.

SUBROUTINE COMP1(CARM1,CF1,CFORC1,CLENGT,DELTC1,
& FU,I,PART,THETA,W)

DIMENSION CARM1(200),CF1(200),DELTC1(200),SIGMAC(200)

CFORC1 = 0
DO 1000 N = 1,100
IF(N.GT.I)GO TO 1100
DELTC1(N) = N*PART*TAN(THETA)
CARM1(N) = N*PART
SIGMAC(N) = 2000.*(FU/64.1)*DELTC1(N)
IF(SIGMAC(N).GT.((FU/64.1)*24.))
& SIGMAC(N) = (FU/64.1)*(- 21.2 + 45.4*DELTC1(N)
& - 241.0*DELTC1(N)**0.5
& + 310.0*DELTC1(N)**0.333 )

CF1(N) = SIGMAC(N)*PART*W
CFORC1 = CFORC1 + CF1(N)
IF(N.EQ.I)CFORC1 = CFORC1 + (1- 0.5*PART)*SIGMAC(N)*W
GO TO 1000

1100 CONTINUE
CF1(N) = 0.0
CARM1(N) = 0.0

1000 CONTINUE

RETURN
END

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
SUBROUTINE COMP2:
Calculates the compressive force distribution in a
flexible end plate connection using the HAFEZ
compression model.

SUBROUTINE COMP2(CARM2,CF2,CFORC2,CLENGT,DELTC2,
& FU,I,PART,THETA,W)

DIMENSION CARM2(200),CF2(200),DELTC2(200),SIGMAC(200)

CFORC2 = 0
DO 1000 N = 1,100
IF(N.GT.I)GO TO 1100
DELTC2(N) = N*PART*TAN(THETA)
CARM2(N) = N*PART
SIGMAC(N) = 2000.*DELTC2(N)

```



```

C
C
SUBROUTINE TENS1(CLEARL, DELTAE, DELTAP, DELTT1, EP,
& HMAX, I, MUP, PYP, PART, T, TARM1, TEN1, TFORC1, THETA)
C
REAL MUP
C
DIMENSION DELTT1(200), TARM1(200), TEN1(200)
C
J = 100 - I
TFORC1 = 0
DO 1000 N = 1, 100
C
IF(N.GT.J)GO TO 1100
DELTT1(N) = N*PART*TAN(THETA)
TARM1(N) = N*PART
IF(DELTT1(N).LE.DELTAE)TEN1(N) = 2.*EP*PART*T**3*
& DELTT1(N)/CLEARL**3
IF(DELTT1(N).GT.DELTAE.AND.DELTT1(N).LE.DELTAP)
& TEN1(N) = (PYP**2*DELTT1(N)**2/(4.*MUP*CLEARL)
& + 4.*MUP*CLEARL/(CLEARL**2 + DELTT1(N)**2))*PART
IF(DELTT1(N).GT.DELTAP)TEN1(N) = (2.*HMAX*DELTT1(N)/
& CLEARL + 4.*MUP*CLEARL/(CLEARL**2 + DELTT1(N)**2)
& - 4.*MUP*HMAX**2/(CLEARL*PYP**2))*PART
C
TFORC1 = TEN1(N) + TFORC1
GO TO 1000
1100 CONTINUE
TARM1(N) = 0.0
TEN1(N) = 0.0
1000 CONTINUE
C
RETURN
END
C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
SUBROUTINE TENS2:
C
Calculates the tensile force distribution in a
flexible end plate connection using the
KENNEDY-HAFEZ model.
C
SUBROUTINE TENS2(CLEARL, DELTAE, DELTAP, DELTT2, DH, EP,
& HMAX, I, MUP, P, PYP, PART, T, TARM2, TEN2, TFORC2, THETA)
C
REAL MUP
C
DIMENSION DELTT2(200), TARM2(200), TEN2(200)
C
J = 100 - I
TFORC2 = 0

```

```

DO 1000 N = 1,100
IF(N.GT.J)GO TO 1100
C
DELTT2(N) = N*PART*TAN(THETA)
TARM2(N) = N*PART
IF(DELTT2(N).LE.DELTAE)TEN2(N) = 2.*EP*PART*T**3*
& DELTT2(N)/ CLEARL**3
IF(DELTT2(N).GT.DELTAE)
& TEN2(N) = (PYP**2*DELTT2(N)**2*((P - DH)/P)**2/
& (4.*MUP*CLEARL) + 4.*MUP*CLEARL/(CLEARL**2 +
& DELTT2(N)**2))*PART
C
TFORC2 = TEN2(N) + TFORC2
GO TO 1000
1100 CONTINUE
TARM2(N) = 0.0
TEN2(N) = 0.0
1000 CONTINUE
C
RETURN
END
C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
SUBROUTINE TENS3:
C
C Calculates the tensile force distribution in a
C flexible end plate connection using the HAFEZ model.
C
C
SUBROUTINE TENS3(CLEARL,DELTAE,DELTAP,DELTT3,EP,
& HMAX,I,MUP, PYP,PART,T,TARM3,TEN3,TFORC3,THETA)
C
REAL MUP
C
DIMENSION DELTT3(200),TARM3(200),TEN3(200)
C
J = 100 - I
TFORC3 = 0
DO 1000 N = 1,100
IF(N.GT.J)GO TO 1100
C
DELTT3(N) = N*PART*TAN(THETA)
TARM3(N) = N*PART
IF(DELTT3(N).LE.DELTAE)TEN3(N) = 2.*EP*PART*T**3*
& DELTT3(N)/CLEARL**3
IF(DELTT3(N).GT.DELTAE)
& TEN3(N) = (PYP**2*DELTT3(N)**2/(4.*MUP*CLEARL)
& + 4.*MUP*CLEARL/(CLEARL**2 + DELTT3(N)**2))*PART
C
TFORC3 = TEN3(N) + TFORC3
GO TO 1000
1100 CONTINUE

```

[illegible]

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC

C
C
C
C
C
C
C

SUBROUTINE WRITE:

When the TABLE option for output is selected this routine is called to generate the output.

C

```
SUBROUTINE WRITE(DB,DEPTHB,DEPTHHP,DH,E,ENDM1,
& ENDM2,ENDM3,EP,F,FY,FYR,FU,FUB,FUP,G,
& MOMEN1,MOMEN2,MOMEN3, P,T,TABLE,THETA1,
& THETA2,THETA3,TITLE,TOPDIS,W)
```

C

REAL MOMEN1,MOMEN2,MOMEN3

C

```
DIMENSION ENDM1(20),ENDM2(20),ENDM3(20),
& TABLE(5),TITLE(20)
```

```
WRITE(6,100)TABLE(1),TABLE(2),TITLE(1),TITLE(2),
& TITLE(3),TITLE(4),TITLE(5),TITLE(6),
& TITLE(7),TITLE(8),TITLE(9),TITLE(10)
100 FORMAT('1',////,T42,'TABLE ',2A4,/,
& T19,'COMPARISON OF ANALYTICAL PREDICTIONS OF',
& ' MOMENT-ROTATION',/,
& T19,'RELATIONSHIPS FOR A FLEXIBLE END PLATE ',
& 'CONNECTION ',/,/,
& T19,'CONNECTION DESCRIPTION : ',/,+',T19,22(''),
& //,T19,10A4)
```

C

IF(UNITS.GE.2)GO TO 5000

C

```
WRITE(6,200)G,P,E,T,F,DEPTHHP,TOPDIS,DEPTHB,W,DH,
& DB,FYP,FUP,EP,FUB,FY,FU
200 FORMAT(' ',/,T19,'Bolt Hole Gage, G = ',
& F7.1,' inches',
& //,T19,'Bolt Hole Pitch, P = ',F7.1,' inches',
& //,T19,'Bolt Hole Edge Dist., E = ',F7.1,' inches',
& //,T19,'End Plate Thickness, T = ',F7.2,' inches',
& //,T19,'Fillet Weld Leg Size, F = ',F7.2,' inches',
& //,T19,'End Plate Depth, DEPTHHP = ',F7.1,' inches',
& //,T19,'Top Distance, TOPDIS = ',F7.1,' inches',
& //,T19,'Beam Depth, DEPTHB = ',F7.1,' inches',
& //,T19,'Beam Web Thickness, W = ',F7.2,' inches',
& //,T19,'Bolt Hole Diameter, DH = ',F7.2,' inches',
& //,T19,'Bolt Diameter, DB = ',F7.2,' inches',
& //,T19,'End Plate Yield Str., FYP = ',F7.1,' ksi',
& //,T19,'End Plate Ult. Str., FUP = ',F7.1,' ksi',
& //,T19,'End Plate Mod. Elast., EP = ',F7.1,' ksi',
& //,T19,'Bolt Ult. Str., FUB = ',F7.1,' ksi',
& //,T19,'Beam Yield Str., FY = ',F7.1,' ksi',
& //,T19,'Beam Ult. Str., FU = ',F7.1,' ksi')
```

C

GO TO 6000

C

5000 CONTINUE

C

```

      WRITE(6,250)G,P,E,T,F,DEPTH,TPDIS,DEPTHB,W,DH,
      & DB,FYP,FUP,EP,FUB,FY,FU
250 FORMAT(' ',/,T19,'Bolt Hole Gage,          G = ',
      &F7.1,' mm',
      &/,T19,'Bolt Hole Pitch,          P = ',F7.1,' mm',
      &/,T19,'Bolt Hole Edge Dist.,      E = ',F7.1,' mm',
      &/,T19,'End Plate Thickness,        T = ',F7.2,' mm',
      &/,T19,'Fillet Weld Leg Size,      F = ',F7.2,' mm',
      &/,T19,'End Plate Depth,          DEPTH = ',F7.1,' mm',
      &/,T19,'Top Distance,              TOPDIS = ',F7.1,' mm',
      &/,T19,'Beam Depth,                DEPTHB = ',F7.1,' mm',
      &/,T19,'Beam Web Thickness,         W = ',F7.2,' mm',
      &/,T19,'Bolt Hole Diameter,        DH = ',F7.2,' mm',
      &/,T19,'Bolt Diameter,             DB = ',F7.2,' mm',
      &/,T19,'End Plate Yield Str.,      FYP = ',F7.1,' MPa',
      &/,T19,'End Plate Ult. Str.,      FUP = ',F7.1,' MPa',
      &/,T19,'End Plate Mod. Elast.,    EP = ',F7.1,' MPa',
      &/,T19,'Bolt Ult. Str.,           FUB = ',F7.1,' MPa',
      &/,T19,'Beam Yield Str.,          FY = ',F7.1,' MPa',
      &/,T19,'Beam Ult. Str.,           FU = ',F7.1,' MPa')

```

C

6000 CONTINUE

C

IF(UNITS.GE.2)GO TO 5500

C

```

      WRITE(6,300)THETA1,MOMEN1,THETA2,MOMEN2,
      & THETA3,MOMEN3
300 FORMAT(' ',/,T19,'MOMENT-ROTATION RELATIONSHIP :',
      & /,+',',T19,28(' '),
      &/,T19,'KRIVIAK ROT. LIMIT,          THETA1 = ',
      & F7.4,' radians',/,
      & T19,'KRIVIAK MOM. LIMIT,          MOMENT1 = ',
      & F7.2,' inchkips',/,
      & T19,'KENNEDY-HAFEZ ROT. LIMIT,    THETA2 = ',
      & F7.4,' radians',/,
      & T19,'KENNEDY-HAFEZ MOM. LIMIT,    MOMENT2 = ',
      & F7.2,' inchkips',/,
      & T19,'HAFEZ ROT. LIMIT,          THETA3 = ',
      & F7.4,' radians',/,
      & T19,'HAFEZ ROT. LIMIT,          MOMENT3 = ',
      &F7.2,' inchkips',// )
      WRITE(6,301)
301 FORMAT(' ',
      & T19,'          CONNECTION      KRIVIAK      KENNEDY',
      & T19,'          HAFEZ          ',/,
      & T19,' #          ROTATION      MOMENT      HAFEZ ',
      & T19,'          MOMENT ',/,
      & T19,'          ',/,
      & T19,'          (radians)      (inchkips)      (inchkips)',
      & T19,'          (inchkips)',/)

```

C


```

C
C
SUBROUTINE WRITE2(DB,DEPTHB,DEPTHP,DH,E,ENDM1,ENDM2,
& ENDM3,EP,F,FY,FYP,FU,FUB,FUP,G,MOMEN1,MOMEN2,
& MOMEN3,P,T,TABLE,THETA1,THETA2,THETA3,
& TITLE,TPDIS,W)
C
REAL MOMEN1,MOMEN2,MOMEN3
C
DIMENSION ENDM1(20),ENDM2(20),ENDM3(20),TABLE(5),
& TITLE(20),ENDMO(20)
C
XX = 0.
YY = 0.
DO 1500 N = 1,3
WRITE(6,450)TITLE(1),TITLE(2),TITLE(3),TITLE(4),
& TITLE(5),TITLE(6),TITLE(7),TITLE(8),
& TITLE(9),TITLE(10),N
450 FORMAT(10A4,I1)
WRITE(6,400)XX,YY
DO 1000 I = 1,10
IF(I.LE.5)THETA = 0.001*I
IF(I.GT.5)THETA = 0.005 + (I - 5)*0.005
C
C
THETA = I*0.0025
IF(N.EQ.1)ENDMO(I) = ENDM1(I)
IF(N.EQ.2)ENDMO(I) = ENDM2(I)
IF(N.EQ.3)ENDMO(I) = ENDM3(I)
WRITE(6,400)THETA,ENDMO(I)
400 FORMAT(' ',F7.5,F7.2)
C
1000 CONTINUE
IF(N.EQ.1)WRITE(6,400)THETA1,MOMEN1
IF(N.EQ.2)WRITE(6,400)THETA2,MOMEN2
IF(N.EQ.3)WRITE(6,400)THETA3,MOMEN3
1500 CONTINUE
C
RETURN
END

```

Appendix B

Computer Program CDSSSS: Connection Design for Simply
Supported Steel Shapes

The listing of this program is lengthy and is therefore not reproduced here. A copy of it is on record at the Department of Civil Engineering at the University of Alberta, Edmonton, Alberta, Canada.

Appendix C

Design Example Using Standardized Connection Design Tables

9.1 to 9.4

Example - Selection of a flexible end plate connection when the supported member span length and factored load are known.

Given:

W530x82 beam, 9000 mm long.

Factored load, 52 kN/m.

Solution:

- i. From the bottom of page 1 of Table 9.1 the beam lengths which bracket the given length are

7920 mm & 9504 mm

- ii. Calculating the maximum factored moment for the case at hand,

$$\begin{aligned} M_f &= w l^2 / 8 \\ &= 52 \times (9)^2 / 8 \\ &= 526.5 \text{ kNm} \end{aligned}$$

and dividing this value by the factored moment resistance of a W530x82, found at the top of page 1 of Table 9.1 gives the loading ratio to be:

$$M_f / M_r = 526.5 / 558.9 = .942$$

Therefore, it is likely that Table 9.2 is appropriate for use here with a uniformly distributed load set at 95 percent of the ultimate load. From the bottom of page 1 of Table 9.2 the factored loads associated with the two bracketing lengths are:

$$67.7 \text{ kN/m} \quad \& \quad 47.0 \text{ kN/m}$$

which, as expected, bracket the given factored load.

- iii. From pages 2 and 3 of Table 9.2, for the two span lengths, the appropriate connection lengths for the 4 possible plate thicknesses are:

Conn # 1, plate thick. 6 mm, 295 mm & 370 mm,
Conn # 2, plate thick. 8 mm, 220 mm & 295 mm,
Conn # 3, plate thick. 10 mm, 220 mm & 295 mm,
Conn # 4, plate thick. 12 mm, 445 mm & 370 mm,

If the design criteria was to use a connection detail of a depth equal to at least one half of the supported member depth and having the least cost, then Connection # 1 with a plate length of 295 mm would be the choice because its cost at \$141.77 is the least, while meeting the depth criteria. The possibility of premature bottom flange bearing with this, the shorter of the two possible connection details, is acknowledged

and individual designers must justify in their own minds which of the two details will serve better.

If the design criteria was to use a connection detail of a depth equal to at least one half of the supported member depth and having the least moment, Connection # 1 would again be the designers choice because its associated factored moment is about 16 kNm whereas the other connection details induce moments all well above 20 kNm.

- iv. If the 6 mm thick end plate detail is selected the symbolic reference, as taken from page 2 of Table 9.2 is:

FEP(300-21-140-75-35-30-75-6-295) B(825-19-8)

W(480-5)